California State Water Project

Volume II
Conveyance Facilities

Bulletin Number 200
November 1974

State of California
The Resources Agency
Department of Water Resources

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CALIFORNIA STATE WATER PROJECT

Volume II

Conveyance Facilities

November 1974

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FOREWORD

This is the second of six volumes which record aspects of the planning, financing, design, construction, and operation of the California State Water Project. The subjects of the other volumes are: Volume I, History, Planning, and Early Progress; Volume III, Storage Facilities; Volume IV, Power and Pumping Facilities; Volume V, Control Facilities; and Volume VI, Project Supplements.

The State Water Project conserves and distributes water to many of California's urban areas for domestic and industrial use and to agricultural lands for irrigation. It also produces power; provides flood control, water quality control, and new recreational opportunities; and enhances sports fisheries and wildlife habitat.

Construction of the first phase of the State Water Project was completed in 1973 at a reimbursable cost of $2.3 billion. It is expected that another $0.7 billion will be spent during the next decade to construct authorized facilities for full operation.

This volume summarizes various engineering aspects of the State Water Project relating to the system which conveys project water between reservoirs, pumping plants, and power plants. The authorized conveyance facilities total 684 miles in length, most of which are canal with a relatively small proportion in pipeline and tunnel.

The conveyance facilities are divided geographically into three aqueducts: the North Bay, South Bay, and California Aqueducts. The California Aqueduct has two branches, Coastal Branch and West Branch, and six divisions, including the San Luis Division, a joint-use facility of the Department of Water Resources and the U.S. Bureau of Reclamation, which was designed and constructed by the Bureau. Aspects of design and construction common to the entire conveyance system are discussed initially, with more extensive coverage given to the individual aqueducts and their divisions and branches.

This volume describes the design activities leading to and including the preparation of contract plans and specifications and the construction work under the various contracts which produced the completed facilities. The time frame is approximately 15 years, from the late 1950s to the early 1970s.

John R. Teerink

John R. Teerink, Director
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ABSTRACT

This volume is a report on the design and construction of an authorized 684 miles of canals, pipelines, and tunnels which convey, or will convey, the waters of the State Water Project to its water users. First water deliveries were made in 1962 in the Livermore Valley through the partially completed South Bay Aqueduct. In 1972, the Santa Ana Valley Pipeline connection to Lake Perris completed the 444-mile-long California Aqueduct, principal conveyance feature of the Project.

The large volume of flow in the conveyance facilities, the varied nature of the terrain traversed, and the formidable problems posed by subsidence of soils and threat of earthquakes presented engineering challenges unprecedented in their complexity and magnitude. Solutions required the use of advanced technology as well as development of new criteria, laboratory techniques, and testing methods. Computer technology, in particular, allowed design engineers to consider many more alternatives than would have been possible a few years earlier.

This volume presents the design criteria that were established for conveyance facilities based on subsurface exploration and information developed in preliminary and final design periods. Topographic, climatic, and geologic conditions existing throughout the Project also are discussed.

The balance of the volume is devoted to specific geographical segments of the Project. General information is augmented on a local basis, and the completed facilities are described in connection with construction activities of the individual contracts. The facilities generally are described on a north-to-south basis.

When describing similar contracts, repetition is minimized by highlighting changes or specialized local details once a construction procedure has been described.

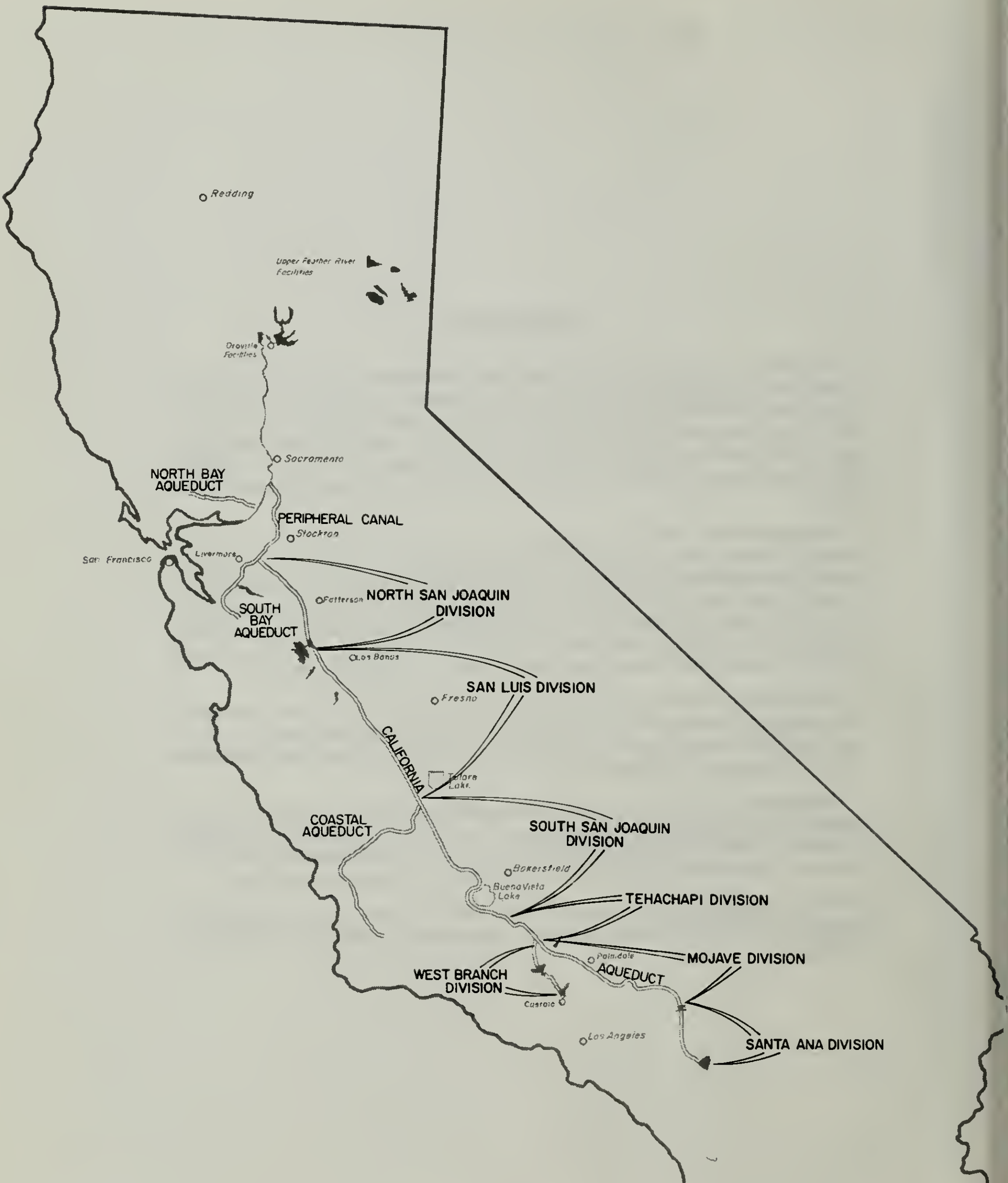

Figure 1. The State Water Project

CHAPTER I. GENERAL

Introduction

The conveyance facilities of the State Water Project (Figure 1) are divided geographically into three aqueducts: the North Bay, South Bay, and California Aqueducts. The California Aqueduct has two branches, Coastal Branch and West Branch, and six divisions.

Upper Feather and Oroville Divisions

The Upper Feather Division of the State Water Project consists of reservoirs without conveyance facilities other than natural streams, except for the Grizzly Valley Pipeline between Lake Davis and Portola which is discussed in Volume III of this bulletin. Similarly, between the Oroville facilities and Clifton Court Forebay, existing river channels are used as conveyance facilities for project water. The river outlet works below Thermalito Afterbay discharges water conserved by Oroville Dam into the channel of the Feather River, where it flows for 50 miles to its confluence with the Sacramento River. The Sacramento River then conveys the water for approximately 77 miles southward to and through the Sacramento-San Joaquin Delta area to West Canal and Old River, from which it enters Clifton Court Forebay.

Delta Facilities

The Peripheral Canal, an unlined canal, is being designed to convey good quality water for the State Water Project and the Central Valley Project about 43 miles around the eastern edge of the Delta to Clifton Court Forebay and to 12 release facilities along the Canal for improvement of Delta water quality and aquatic environment. The canal intake works, which will include a fish screen and flood gates, will be located just below the City of Hood on the Sacramento River. The Canal will be designed to divert a maximum flow rate of 23,300 cubic feet per second (cfs) from the Sacramento River and deliver a maximum of 18,300 cfs to Clifton Court Forebay. A large pumping plant to provide the necessary lift, 4 siphons for major river crossings, and 12 water quality turnouts to the Delta also will be provided.

North Bay Aqueduct

North Bay Aqueduct was designed to serve portions of Napa and Solano Counties north of the San Francisco Bay area. This aqueduct will run in a westerly direction for 26 miles from the northern Sacramento-San Joaquin Delta. Phase I, which is completed, derives an interim water supply from Putah South Canal of the U.S. Bureau of Reclamation's Solano Project. An interim pumping plant lifts water from the terminal reservoir of the Solano Project just north of the town of Cordelia to Cordelia Surge Tank. From the Surge Tank, a pipeline conveys the water by gravity 4.2 miles to the terminal facilities of North Bay Aqueduct, a large steel storage tank south of the City of Napa.

Phase II, presently scheduled for completion in 1980, will originate at Lindsey Slough in the northwestern part of the Delta. Project water will be conveyed westward at a maximum flow rate of 115 cfs by canal and pipeline, with the assistance of three pumping plants, to a junction with the existing Phase I pipeline at Cordelia Surge Tank. Turnouts to Solano County will be provided.

South Bay Aqueduct

South Bay Aqueduct is a 44.7-mile-long conveyance system which provides service from the southern portion of the Sacramento-San Joaquin Delta to various South Bay areas and to the City of San Jose. It begins at Bethany Reservoir, an enlarged section of the California Aqueduct, 1 mile south of Delta Pumping Plant. South Bay Pumping Plant pumps a maximum flow of 300 cfs from the Reservoir up and over the eastern ridge of the Diablo Range. The water then is conveyed by gravity through a system of canals, tunnels, and pipelines across Livermore and Santa Clara Valleys to a terminal storage tank east of San Jose. The system provided initial deliveries in 1962 and has been fully operational since 1965.

California Aqueduct

The California Aqueduct (Figure 2) is the principal conveyance system of the State Water Project. This aqueduct originates in the southern Sacramento-San Joaquin Delta and extends for 444 miles southward to Lake Perris, near the City of Riverside. Overall, it consists of a series of canals, pipelines, tunnels, pumping plants, and power plants which convey water from an elevation near sea level in the Delta to service areas and terminal reservoirs in Central and Southern California. It reaches a maximum invert elevation of 3,465 feet in passing through the Mojave Desert.

Within the San Joaquin Valley, essentially the California Aqueduct is all canal and comprises three divisions: North San Joaquin, San Luis, and South San Joaquin. The Coastal Branch turns off from the South San Joaquin Division to serve portions of Kings and Kern Counties and, ultimately, San Luis Obispo and Santa Barbara Counties. The Aqueduct is further divided into the Tehachapi Division, which crosses the Tehachapi Mountains; the Mojave Division, which skirts the Mojave Desert; and the Santa Ana Division, which crosses the San Bernardino Mountains and terminates at Lake Perris. The West Branch of the California Aqueduct starts at the south end of the

Figure 2. Aerial View—California Aqueduct

Tehachapi Division and continues southwest from the Tehachapis to Castaic Lake.

North San Joaquin Division. The North San Joaquin Division extends from the Sacramento-San Joaquin Delta southward 68.4 miles to O'Neill Forebay of the San Luis Joint-Use Facilities.

The design capacity is 10,300 cfs between Delta Pumping Plant and Bethany Reservoir, the take-off point of South Bay Aqueduct. From Bethany Reservoir to O'Neill Forebay, the capacity is 10,000 cfs. Within this division, the Aqueduct is all concrete canal. Service to water users is limited to four small turnouts for a local water district.

San Luis Division. The San Luis Division is a federal-state joint-use facility of concrete-lined canal extending 106 miles from the north end of O'Neill Forebay to Kettleman City. All of the facilities within this division were designed and constructed by the U. S. Bureau of Reclamation. Dos Amigos Pumping Plant, 16 miles downstream of O'Neill Forebay, provides 113 feet of lift necessary for gravity flow through the remainder of this division. As a joint-use facility, the canal also conveys Central Valley Project water to the federal water service area it traverses. There are no turnouts in this division for State Water Project services, but turnouts are provided for the federal service area. Design capacity in this division ranges from 13,100 cfs leaving O'Neill Forebay to 8,350 cfs (7,050 State) at Kettleman City.

South San Joaquin Division. This division is a concrete-lined canal 121 miles in length. It continues the southeasterly flow of water adjacent to the foothills of the Coast Range to the division terminus at A. D. Edmonston Pumping Plant. Buena Vista, Wheeler Ridge, and Wind Gap Pumping Plants provide a total lift of approximately 956 feet to convey the water through the foothills. A large portion (approximately one-third) of project water is diverted by water users within this division, nearly all for agricultural purposes. The design capacity varies from 8,100 cfs below the check gates at Kettleman City to 4,400 cfs at A. D Edmonston Pumping Plant.

Coastal Branch. The California Aqueduct ha[s] capacity for future deliveries to portions of San Lui[s] Obispo and Santa Barbara Counties. These countie[s] will be served from a branch of the main aqueduc[t] commencing 12 miles south of Kettleman City an[d] terminating about 96 miles west and southwest nea[r] the City of Santa Maria on the California coast. Pres[-]ently, it is planned as a combination of canal and pip[e]line and will cross the Coast Range with the aid of fiv[e] pumping plants, two of which are completed an[d] three additional ones scheduled for completion i[n] 1982.

Constructed facilities consist of 14 miles of concrete-lined canal and two pumping plants which raise the water through the fringe of the foothills. For the present, the local demands of water contracting agencies in Kings and Kern Counties are served. The diversion capacity of the Coastal Branch is 450 cfs.

Tehachapi Division. The California Aqueduct crosses the Tehachapi Mountains through an 8.5-mile system of tunnels and siphons which daylight at critical points. The overall length of the Division from A. D. Edmonston Pumping Plant through Tehachapi Afterbay is 10.6 miles. Probably no phase in the planning and design of the State Water Project was given more consideration than the method by which project water was to be conveyed across this mountain barrier into Southern California. Once the route was selected, several alternative schemes were investigated for pumping the water more than 1,900 feet up the face of the Tehachapis.

Water is pumped by A. D. Edmonston Pumping Plant in a single 1,926-foot lift to invert elevation 3,090 feet, from which elevation it flows by gravity across the Tehachapis through four tunnels. The four tunnels are connected by siphons or cast-in-place pipe sections which provide access at the critical fault crossings as well as operational access to the tunnels. The design capacity of the Tehachapi Division is 5,360 cfs.

The tunnels terminate on the southern side of the Tehachapi Mountains at Tehachapi Afterbay. From this point (Figure 3), the West Branch of the California Aqueduct proceeds in a southwesterly direction to Castaic Lake, while the main California Aqueduct continues southeasterly across Antelope Valley.

West Branch. Construction of the West Branch Division is being staged. The first stage provides interim conveyance features between the end of Quail Canal and Pyramid Lake limiting its present conveyance capacity to 900 cfs. This 5.5-mile-long interim channel, Gorman Creek Improvement, parallels Interstate Highway 5 and Gorman Creek. The second stage will replace the interim Gorman Creek Improvement facilities with the Peace Valley Pipeline and the 157-megawatt Pyramid Powerplant, which are to be operational in 1983.

The Quail Canal portion of the West Branch is a concrete-lined canal of 3,129 cfs capacity. Oso Pumping Plant, at the beginning of this reach, provides the lift required for gravity flow through the remainder of the Branch. Quail Lake presently is a 5,020-acre-foot storage and regulating pool.

Between Pyramid Lake and the terminus of West Branch, Castaic Lake, water is conveyed by the 7.15-mile Angeles Tunnel, which has a design capacity of

Figure 3. Bifurcation of "Main Line" California Aqueduct and West Branch

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Figure 4. Aerial View—Tehachapi Crassing

18,400 cfs. This is the largest capacity of all the conveyance facilities now in operation, and it serves the Castaic pumped-storage power development which is a joint facility of the State and the Department of Water and Power of the City of Los Angeles. The Tunnel was designed and constructed by the State. The power and pumping facilities and appurtenances were designed and constructed by the City. The surge tank was designed by the City and constructed by the State.

Mojave and Santa Ana Divisions. The Mojave Division, with a capacity of 2,388 cfs leaving Tehachapi Afterbay, continues the California Aqueduct southeastward and conveys the water through the Antelope Valley-Mojave Desert 102 miles, mostly in lined canal, to Silverwood Lake, a regulating and storage reservoir in the foothills of the San Bernardino Mountains. Pearblossom Pumping Plant, about midway in this division, provides a 540-foot lift. The Santa Ana Division starts with the San Bernardino Tunnel, 3.8 miles in length with a capacity of 2,020 cfs, which conveys the water southward from Silverwood Lake through the San Bernardino Mountains to Devil Canyon Powerplant. The Santa Ana Division continues with the 28-mile Santa Ana Valley Pipeline with a design capacity which decreases from 469 to 444 cfs at Lake Perris, the terminal reservoir of the State Water Project.

Rainfall and Hydrology

A wide range and pattern of precipitation is associated with the topography traversed by the Project's aqueducts. It varies from 50 or more inches in the high Sierra Nevada to only a few inches in the desert areas. However, as widespread as this variance is, one common denominator is that, with the exception of occasional thunderstorms, precipitation occurs only during the period from October through May.

The rainfall period is, on occasion, subject to a distinctive pattern of moist warm air from the central Pacific that produces lengthy storms with periods of high-intensity rain and snow. These storms are responsible for the heavy runoff and floods that occur, particularly in Northern California. Any portion of the State, however, can experience local storms with intense rainfall and local flooding.

All of the streams encountered on the western side of the San Joaquin Valley and south of the Tehachapis are intermittent, flowing only during, and shortly after, storm periods.

Geology and Soils

Geology

Geologic conditions vary greatly throughout the Project. Many different rock types are encountered in complex geologic structures.

Through the North and South San Joaquin and San Luis Divisions, much of the conveyance system is in the alluvium of the San Joaquin Valley. However, parts of the canal cross into Tertiary siltstones, shales, and sandstones in the Coastal Range foothills. Folded, Tertiary, sedimentary rocks also are encountered at Buena Vista Hills, at Wheeler Ridge, and in the vicinity of A. D. Edmonston Pumping Plant.

South of the San Joaquin Valley, the Project penetrates crystalline rocks in the Tehachapi Mountains (Figure 4), a faulted mountain block containing granitic and metamorphic rocks.

Southeasterly of the Tehachapi Mountains, the California Aqueduct is mostly in alluvium on the floor of the Mojave Desert but also cuts into the foothills bordering the Desert, particularly west of Pearblossom Pumping Plant. In the foothills, weathered granitic rocks, metamorphic rocks, and some Tertiary sandstones and siltstones are encountered. Granitic and metamorphic rocks are penetrated by tunnel in the San Bernardino Mountains. Throughout the remainder of the Santa Ana Division to Lake Perris, the pipeline is mostly in alluvial deposits but does encounter a few granitic rock formations in the vicinity of Box Springs.

On the West Branch, folded sandstones, siltstones, and shales of the Ridge Basin group are encountered.

Exploration

A large amount of geologic exploration was required for various kinds of facilities in the widely diverse array of geologic conditions. Once the exploration of possible alternative aqueduct routes and plant sites was completed, additional exploration was made to provide sufficient detailed geologic information for design purposes.

During construction, a carefully detailed record was made of geologic conditions encountered, and a final geologic as-built report was prepared for each contract.

Seismicity

Aside from the usual problem of finding a suitable foundation for structures, there were some unusual geologic considerations. One of the foremost of these was the seismicity of the region traversed by the State Water Project. The northern portion of the Project in the Upper Feather River and at Lake Oroville is in a region that is seismically quiet but, through the San Joaquin Valley and into Southern California, the conveyance system roughly parallels the San Andreas fault, the source of many large earthquakes. The California Aqueduct crosses the San Andreas fault at four places: Quail Lake, Anaverde Valley, Barrel Springs near Palmdale, and Devil Canyon Powerplant. Other major fault crossings are the Garlock fault zone in the Tehachapi Mountains and the San Jacinto fault south of the San Bernardino Mountains. The West Branch also crosses the San Andreas fault, and South Bay Aqueduct crosses the Calaveras fault. In addition to these major faults, numerous minor faults are crossed by various features of the Project.

The proximity to major active fault zones poses the hazard of damage by ground shaking from large earthquakes occurring along these zones. Where the conveyance systems must cross active faults, there is the possibility of damage by displacement along the fault. To assess the seismic hazard, earthquake hazard reports were prepared for each major structure. The Department of Water Resources' Consulting Board for Earthquake Analysis aided in establishing design criteria for each structure.

Geodimeter surveys were made across the San Andreas and Garlock fault zones to measure fault creep. Special survey nets called quadrilaterals also were installed to measure fault creep at or near crossings of the San Andreas, Garlock, and San Jacinto fault zones. Strong-motion seismographs were installed several years before construction of many key structures to monitor strong ground motions. For a more detailed discussion of the seismic investigation program, see Volume VI of this bulletin.

Subsidence

Ground subsidence (Figure 5) was another special geologic consideration. Two types of subsidence are common in the San Joaquin Valley: deep subsidence, caused by extraction of ground water, and shallow subsidence, caused by the collapse of the structure of surface soils when saturated. The subsidence caused concern, because usually it occurred in reaches traversed by canal, where any subsidence could drastically change the nearly flat gradient in the canal. In order to identify areas where shallow subsidence could occur, field test plots were developed and laboratory analyses were conducted. Areas identified as susceptible to shallow subsidence then were saturated by ponding prior to construction, thereby forcing most of the shallow subsidence to occur before construction.

First-order level lines were surveyed for a number of years to identify the areas where deep subsidence was occurring. From these data, future subsidence rates were estimated and additional freeboard built into the canal to compensate for subsidence in future years.

Landslides

There were many high cuts, deep intake channels and plant bowls, and side hill cuts along the entire Project. Around many cuts, slope indicators were installed to monitor the behavior of cut slopes as the excavation progressed so that stability could be checked. Landslides were particularly troublesome in some of the foothill areas traversed by the North San Joaquin Division. Side hill cuts undercut bedding planes in sedimentary rocks and caused landslides which had to be corrected during the construction period. The design criteria of minimizing excavation by establishing reasonably steep cut slopes and correcting landslide failures where they occurred proved to be justified.

Construction Materials

Extensive exploration programs were conducted to locate suitable embankment materials along the canal excavation. When suitable materials were not available in the canal excavation, other nearby sources

Figure 5. Ground Subsidence

were located. If distribution of suitable embankment materials along a contract reach was poor, mass diagrams were sometimes developed to determine the best haulage patterns. Special studies were made of aggregate sources along the Project to determine the availability of concrete aggregate. Sources of riprap also were identified when needed. Sources of water suitable for construction purposes were identified. In the South San Joaquin Division, two deep high-yield wells were constructed by the Department to provide water for shallow subsidence ponding operations.

Gypsum

The climatic and geologic environment in the San Joaquin Valley is conducive to the formation of gypsum in surface soils. Gypsum is readily soluble and when exposed to water leaches out of the soil, causing a reduction in soil volume with damaging settlement. Special techniques for testing in the field were developed and used in the canal construction between Tupman Road and Buena Vista Pumping Plant. Soils with a gypsum content of 10% or greater were overexcavated and replaced with suitable compacted material.

Expansive Clays

Expansive clays were encountered from place to place along the conveyance system. If allowed to remain in a canal bank adjacent to the linings, the clays would absorb water from the canal, expand, and damage the canal lining. Such clays generally are fat montmorillonite-type clays whose field-moisture content is considerably below full saturation. Such zones of expansive clay were identified in part by exploration before construction and in part by field inspection of the canal prism after excavation. Expansive clays were overexcavated and replaced with compacted sublining of suitable material.

Design Concepts and Criteria

Concepts

The science and art of hydraulic design of open channels, although empirical or judgmental to a large degree, have rather well-tested rules from experience for low-volume flows. However, the flow capacities which were to be carried by the California Aqueduct in its upper reaches, 10,000 cfs or more, were beyond experience for this kind of channel and also beyond the prudent range for extrapolation of known data. Therefore, initial calculations required assumptions which, for final design, necessitated further investigation and laboratory substantiation.

The California Aqueduct is a complex nonlinear system which operates normally under unsteady-state conditions. Its hydraulic analysis hinges on an understanding of transient flow phenomena and their influence on the operation and control of the entire system.

The State Water Project is operated on what has been named the "controlled volume concept" as discussed more fully in Volume V of this bulletin. For the Aqueduct, this concept required that a sufficient number of check structures be provided to accommodate rapid flow changes with a minimum of water surface fluctuation. Also, it required that an automatic control system be installed for regulation of the check-structure radial gates to keep all reaches of the Aqueduct checked "full". The control system further provided the means to isolate aqueduct reaches in the event of an emergency condition such as a landslide into the canal.

The adoption of the controlled volume concept had several advantages, such as: (1) elimination of emergency wasteways, (2) minimization of the need for regulating reservoirs and storage reservoirs, (3) preservation of canal lining integrity through isolation of the pool or pools which will be affected by emergency drawdown conditions or loss of water through rupture, (4) reduction of adverse effects resulting from unscheduled flow changes through delivery structures, and (5) lower pumping costs by utilizing off-peak power rates.

Two additional concepts which directly influenced design and construction of the conveyance facilities were: (1) the project conveyance as a whole was to be designed and constructed as a utility with a high degree of structural integrity and performance capability, and (2) the Aqueduct was to be considered as a continuously operating utility.

An additional influencing factor was the adoption of an architectural motif. During the formulation period, a motif was developed to present an acceptable and pleasing image to the public and also to convey a unifying appearance identifiable throughout the Project. Thus, the check structures and their control buildings were painted or trimmed with the same colors used for pumping plants and other buildings.

For the conveyance facilities, an important architectural consideration was the treatment of spoil areas. Spoil areas were to be shaped and finished in configuration and treatment so they would be esthetically pleasing and would blend in with the natural surroundings. The architectural design is more fully discussed in Volume VI of this bulletin.

Special Studies

A special study to evaluate unsteady flow conditions in the Aqueduct was made by the Department of Water Science and Engineering, University of California at Davis. The flow conditions studied were those which may be encountered either in the course of the normal operation of the system or from emergency situations involving rapid closures of gates, pumping plant shutdown, or canal failures.

For the purpose of this study, two principal areas of investigation were defined. The first area consisted of a study of the interaction between the various canal reaches, tunnels, pipelines, and pumping plants of the system for determining how to best control the interaction. The second area provided a study of unsteady-state flow phenomena, primarily in open channels. The studies involved (1) a general schematization of the system, whereby the main components were separated to facilitate their functional description by means of mathematical models; (2) the description of flow in the various components with suitable mathematical equations; (3) development of a complete numerical electronic computer program for the calculation of a variety of steady and unsteady-state situations; and (4) the numerical anaylsis of two of the main components of the Aqueduct for a few specific unsteady flow cases.

The overall study was backed by scale model tests at the University's hydraulic laboratory and by full-scale field tests made with the Delta-Mendota Canal. The results of the study were used to determine flow change limitations and the types of controls needed and to set operational criteria. The mathematical equation and operational programs are still being used to investigate special flow conditions in the Aqueduct.

Aqueduct Lining

One of the early design decisions was to construct a concrete-lined canal rather than a compacted earth-lined section. The relative advantages or disadvantages of a concrete-lined canal versus earth-lined canal are numerous.

The principal advantages of an earth-lined canal are: lower initial cost, less concern with differential settlement, and less threat of lining failure from back-pressure under drawdown conditions. The principal disadvantages of earth lining are: loss of water through seepage, loss of hydraulic head from increased friction losses, lower design velocities to avoid erosion, and greater maintenance requirements.

Because of lower head loss, pumping costs, greater reliability, minimal maintenance, and lesser seepage losses, it was decided that all of the canal would be concrete-lined. The 3-mile unlined intake channel to Delta Pumping Plant and the proposed Peripheral Canal are exceptions, because their prisms are located partially or wholly below sea level in a tidal estuary.

The lining selected was unreinforced concrete with a thickness based on the length of the side slope of the canal section. Lining thicknesses were to be a minimum of 2 inches in thickness, 3½ inches for side slopes from 15 to 30 feet, and 4 inches for longer slopes. Asphaltic concrete, shotcrete, or pneumatically applied mortar, as well as various types and thicknesses of plastic membranes, were considered as lining materials. Asphaltic concrete was the only alternative type given serious consideration but was ruled out due to uncertainties regarding the life of the material and its reaction to aquatic growth.

During construction of the Alameda Canal reach of the South Bay Aqueduct, various methods of use and thicknesses of plastic membranes were tried as underliners on an experimental basis. Construction problems encountered, such as tearing of the membrane and sloughing of the freshly placed concrete down the slick surfaces of the membranes, and inconclusive test data as to advantages resulted in sublining not being adopted for the California Aqueduct.

Reinforced concrete was not considered necessary since the lining is not ordinarily a structural member under stress. The use of reinforced concrete was to be considered an acceptable substitute where swelling soils were encountered as an alternative to the practice of removing these soils and replacing them with compacted embankment; however, it was never used in the California Aqueduct.

Other design considerations in the use of unreinforced concrete as a lining material are the shape and spacing of transverse and longitudinal grooves to obtain controlled contraction cracking and the type of sealant for the grooves. A study made of lining slab thickness and jointing resulted in the following recommendations: (1) the groove dimensions were to be as illustrated on Figure 6; (2) the first longitudinal groove was to be 6 feet down the slope from the top of the lining; (3) additional longitudinal grooves were to be placed at distances not to exceed 12 feet for 4-inch lining or not to exceed 36 times the thickness of the lining, except for 3½-inch lining, where the maximum spacing was to be 10 feet; (4) transverse grooves were to be spaced for distances not to exceed 12 feet for 4-inch lining, 10 feet for 3½-inch lining, and in other cases 36 times the lining thickness and, in all cases, were to be continuous for the section; (5) because of the difficulty in making a sharp angular change with concrete, the intersection of the side slope and the invert was to be rounded. The radius of curvature was to be 3-foot maximum for canals less than 10 feet in depth and 4-foot maximum for canal depths of 10 feet or more; and (6) as the coefficient of shrinkage of the concrete used was expected not to exceed 0.04%, expansion joints were not required. Expansion joints, however, were used where the lining abutted permanent structures, such as transitions, checks, turnouts, and control buildings. If water tightness was required, waterstop or sealant was to be used.

To prevent seepage behind the lining, a horizontal lip of 12 inches was required at the top of the lining and was used to feather the leading edge of the earth berm above the lining.

A study also was made of the type of sealant to be used for contraction joints and the requirements for sealant installation. The sealer or joint filer sought was one with characteristics which would prevent leakage, be immune to the attack of aquatic growth, and last the life of the Project. The number of materials available for use as sealants was growing rapidly. The effectiveness and quality of many of these products varied with each batch produced, and care had to be exercised in their selection and usage.

As the hydrostatic head varied, the same type of

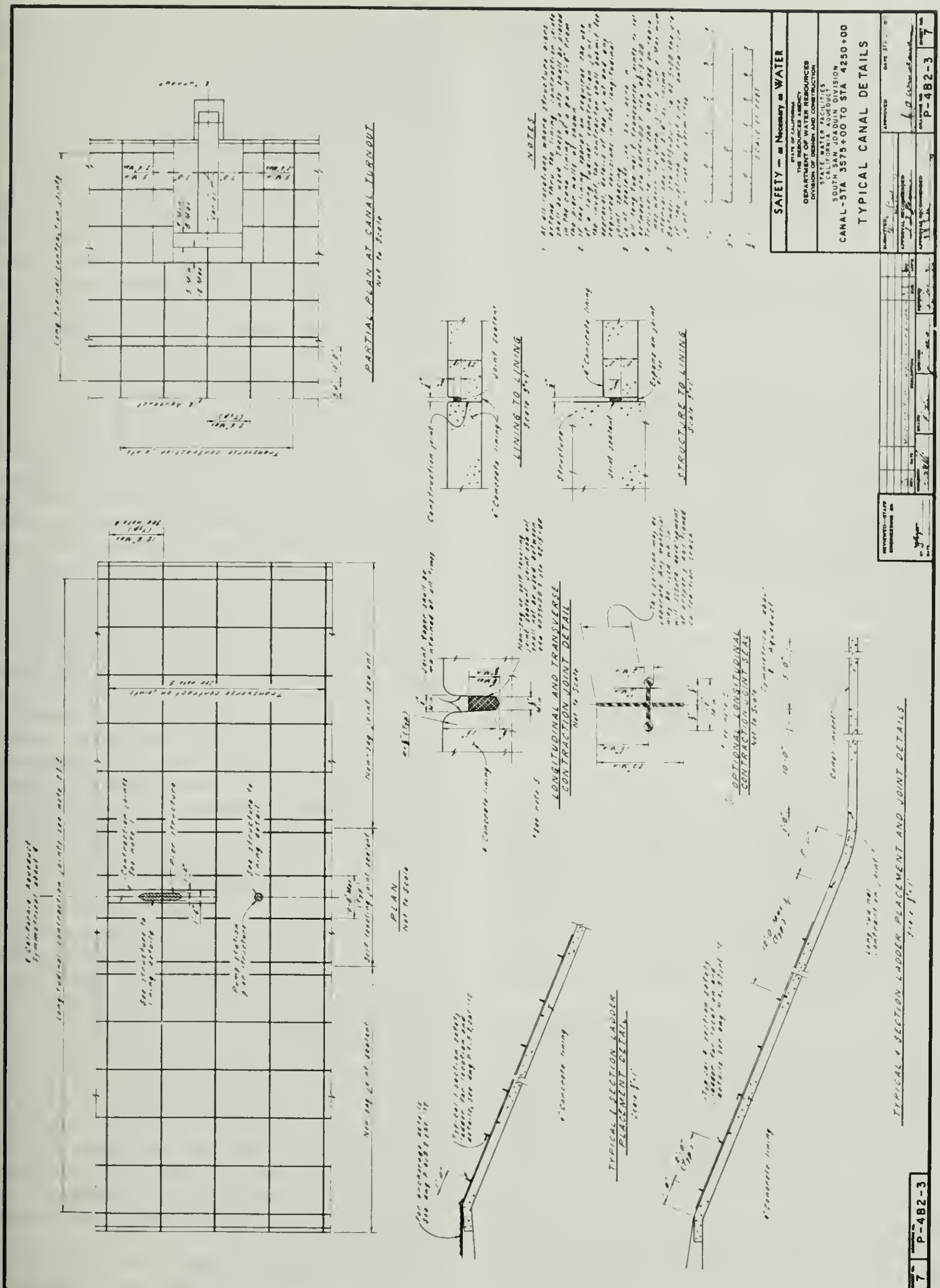

Figure 6. Groove Dimensions

sealant was not required for all locations. Also, the cost of sealant increased sharply as the percentage of synthetic rubber increased, and considerable savings could be realized by specifying different types of sealants. For large sections, the use of three different sealants was given serious consideration. Requirements specified for sealants were: (1) for hydrostatic heads of 10 feet or less, joint fillers of asphalt-rubber mastics were recommended, and the joint was to be completely filled with the compound; (2) for hydrostatic heads of more than 10 feet, elastomeric sealants were to be used. Joints would be filled from the bottom to a minimum depth of sealant of ⅝ of an inch and a maximum depth of ¾ of an inch; (3) sealants were to be used in areas of expansive clays and pervious materials; (4) sealant on side slopes was not to be applied until a few months before the canal section was to be filled. However, grooves in the invert, to avoid cleaning problems, were to be filled soon after curing of the concrete. Exposure of the sealant to sunlight would, in this case, be minimal to avoid deterioration of the sealant; (5) grooves to which sealant was applied were to be sandblasted clean and be dry at the time of sealer application; and (6) preformed groove fillers and shapes inserted to supplant the grooves were not considered satisfactory to provide a leakproof joint.

It was stressed that the prime requisite for a first-class lining was the attention to workmanship which was applied throughout the steps required. This requirement must be backed by a clearly written set of specifications, proper selection of materials and equipment, and an adequate inspection and testing program.

Capacity Requirements

The size of any conduit in the State Water Project was to be the size required for maximum delivery. As the Project's operation is based on the controlled volume concept, this meant all deliveries would be based on the aqueduct reaches remaining full with minimum drawdown to provide water deliveries.

The total capacity required was to be the sum of: (1) maximum water deliveries at the water user's turnouts, (2) operational losses, and (3) contingency capacity required.

The water supply contracts between the Department and the various water users specify the maximum annual entitlement. The delivery of project water was assumed to be on a uniform basis in most cases but could vary from month to month depending on peaking limitations. Agricultural water has a higher peaking rate than municipal and industrial water. To provide flexibility, while controlling the fluctuation to prevent expensive overcapacity for much of the year, the maximum monthly demand was established in the water contracts as 18% for agricultural water and 11% for municipal and industrial water of the annual entitlement on a monthly average basis.

Within a reach serving more than one water user, the Department cannot predictably balance the maximum demands against each other. In such a case, storage or regulation reservoirs do not change the demand but only provide a possible source of the flow. Therefore, the maximum capacity of the conduit needed for water deliveries within a reach is the sum of the maximum monthly demands for all users within that reach, plus downstream requirements.

Operational losses are those computed for the project system only. Losses in the service areas are reflected in the demands by the water users. Losses to spill from refusal of water by a water user were predicted to be low, as intermediate or terminal reservoirs within the system should absorb most of the excess. Three percent was the maximum allowance to be used for this loss. Operational losses have not been experienced during project operations.

The measurement of deliveries at turnouts was to have an error not to exceed 2%. For a system with several turnouts sufficient to change the capacity of the Aqueduct, as in the South San Joaquin Division for example, these measurements should have a balancing effect. For canal sizing purposes, the capacity allowance for measuring error was assumed to equal one-half of the expected measuring error.

Seepage losses should be low, but some loss could be expected through small cracks and joints. Since the lining was placed on a fairly tight subbase without any special treatment or sublining, a value of 0.07 of a cubic foot of seepage per day per square foot of wetted perimeter was used for design purposes. Actual seepage losses have not been measured, but no unexpected problems due to seepage have been experienced.

Evaporation losses occurring within canal sections are significant. The canal water surface area, the arid climate through which the Aqueduct passes, and the reservoir water surface areas combine to require consideration of this loss. Evaporation loss was estimated to be 5 feet of water proportioned over the year with 0.95 of a foot occurring in July and 0.06 of a foot occurring in January.

Contingency capacity was based on loss of flow during an outage of major proportions, such as the complete disruption of a conveyance or the total loss of a pumping plant's capabilities. The duration for such an outage is speculative; however, temporary or bypass facilities should be constructed within a month's time. Seismic action appeared the most likely catastrophic event which would produce such an outage. Outages of more minor consequence could occur during early usage of the facilities from bank or foundation failures. It was anticipated that these outages would reveal themselves during the years of low water demand and, therefore, would not require contingency capacity. Partial loss of pumping capacity could be made up by standby units.

Theoretically, the quantity of flow which should be available from storage during a major outage was th calculated capacity times the period of the outage, or a month's downstream supply. This should be made up well before the next outage, because it would be only a small portion of the flow during the several years which would probably elapse between outages. As the probability for successive major outages was highly speculative, prudent practice dictated that this emergency storage be replaced well before the theoretical or speculative time frame had elapsed. This practice also would maintain a better operating potential for the system.

Conveyance facilities downstream from the San Joaquin Valley were designed for capacities equivalent to the larger of (1) the maximum reach capacity required to permit downstream deliveries on demand in conjunction with regulatory storage, or (2) the capacity required to convey downstream water on a continuous basis plus 7½%. This allowed reserve capacity during years of maximum demand to replace a month's outage within about four weeks, depending on the variation in agricultural demand in the San Joaquin Valley.

The size of the California Aqueduct was not adjusted to new dimensions immediately downstream of a turnout where a reduced configuration could provide the necessary capacity. Otherwise, in areas of heavy demand, frequent dimensional changes would occur which would add to the construction costs by requiring a variety of specialized expensive machinery, such as trimmers and paving jumbos. These potential changes were accumulated to the limit of a contract reach or other suitable location.

The capacity of a portion of the conveyance facilities was established by other means that have been discussed. For example, the capacity of the North San Joaquin Division aqueduct was established through the Burns-Porter Act at not less than 10,000 cfs. The minimum established was used as the design capacity because it provided sufficient outage or reserve capacity.

Overallowance Capacity

In addition to other capacity requirements, all conveyance facilities were sized with a contingency capacity factor of 5%. The required design capacities, as mentioned elsewhere in this volume, were increased by 5% for conveyance sizing purposes. This overallowance factor assured that deliveries could be made even though the roughness factors selected for design were improper or that the roughness changed with time due to marine growth, sediment deposits, or surface deterioration. This contingency reserve was not to be considered as increased capacity in the conveyances but only to assure that the required flow would be realized. When design flow rates in the various canal reaches are experienced and careful measurements can be made of actual flow gradients, the real roughness values and canal capacities can be determined.

Control Requirements

There are four ways in which normal flow in the conveyance facilities will vary. These are: (1) change in pumping or generating rate, (2) releases from a storage or regulating reservoir, (3) deliveries from turnouts, and (4) the setting of check gates or valves within a section of the conveyance facilities. In regard to canals, the location and spacing of checks are of primary importance.

Check locations were selected to perform the five following functions:

1. Maintain a minimum water surface fluctuation to prevent damage to the canal lining due to high ground water.
2. Maintain a minimum water surface elevation for turnouts.
3. Isolate reaches of a given canal should emergency repair or emergency maintenance be required.
4. Contain the water in transit in specific reaches when a shutdown of the system occurs.
5. Provide storage for local delivery operation.

Operating Roads and Bridges

A surfaced all-weather road was required along all canal reaches. Roads on both sides of the canal were required where the distance from the inside edge of the roadway to the bottom of the far canal side slope exceeded 36 feet.

Operating roads were required to be between 2 and 4 feet above the top of the canal lining, and to drain away from the canal. A windrow was placed on the canal side to prevent the wheels of vehicles from kicking loose material into the canal and to indicate possible entry of a vehicle into the canal. A maximum grade of 10% was permitted. One of the two roads was designated as the primary operating road with the intention that it would be utilized for maintenance patrolling and through-traveling. The primary roadway received better surfacing and appurtenances, such as cattle guards where required for convenience. Lockable gates were provided at all crossroad access points. Where possible in high embankment areas, the primary road was placed on the downslope side to provide better inspection of the highest embankment. An overhead clearance of 15 feet was required over all operating roads.

Bridges with a 16-foot minimum roadway built to H-20 loading specifications were required at primary operating road locations subject to flooding where alternative access was not available using local public roads. In addition, an operational bridge or alternative vehicular crossing of the canal generally was required at a maximum interval of 4 miles.

Cross Drainage

Type. Cross drainage required a large design effort. The policy was established early that cross drainage would not be introduced into the canal because of water quality considerations except in the San Luis

Division. The cross-drainage flow rate and relative elevations of the canal and the watercourse required that each drainage crossing be given individual study. Where possible, adjacent watercourses were combined to obtain a single crossing. However, additional right of way, flow easements, and legal problems restricted the use of this method. Cross drainage was accomplished through a choice of (1) overchutes, (2) culverts, (3) siphon undercrossings, and (4) drain inlets.

Overchutes were favored where the drainage channel invert was high enough that it could be easily carried across the canal prism. Overchutes usually were supported by a single pier to minimize head loss in the Aqueduct.

Culverts were used where inlet and outlet conditions required, such as at high canal embankment locations. Some clogging by sediment or debris from the variable flow conditions was expected. Periodic maintenance for debris removal sometimes was the only practical solution.

The choice between a culvert undercrossing for drainage flows or a siphon undercrossing for canal flows largely was based on the volume of flow in the drainage course and the disturbance of the existing channel equilibrium with a culvert undercrossing. Large-volume cross flow was a prime consideration favoring a canal siphon.

Except in the San Luis Division, drain inlets were used only for small flows and then only when the quality of the water and its debris content were not adverse factors.

Flow Requirements. Maximum flow conditions in the intermittent watercourses encountered are not subject to precise determination. Published or formal flow records were seldom available. Field inspection, probing of local residents' knowledge, and records or calculations of adjoining municipalities or governmental agencies were avenues used to develop flow conditions. The manual "California Culvert Practice," second edition, published by the Division of Highways (now the Department of Transportation), was used as a guide for drainage areas of less than 10 square miles in area. As the design and construction of Interstate Highway 5 paralleled in both time and location that of the California Aqueduct through the San Joaquin Valley, much valuable drainage information was shared between the Department and the Division of Highways.

The cross-drainage design accommodated the following criteria:

1. The 5-year storm flow was to be confined inside the natural channel or inside purchased right of way or flow easements.

2. The 100-year storm flow was to be passed with no damage to the channel or structure although some flooding was to be permitted. Design freeboards for the structure and embankments were to be provided for this condition.

3. The 500-year storm flow was to be passed with no danger to the Aqueduct. Damage to the downstream channel and some damage to the outlet structure could be permitted. The upstream embankment should have a minimum freeboard of 3 feet, measured from maximum water surface elevation to the top of the structure.

For those drainage sites where temporary ponding could be tolerated, the canal embankment was required to be protected from any danger of saturation from the ponded water.

Channel Equilibrium. Consideration of the effects of the cross-drainage facility on the natural drainage channel was included in the design of the cross-drainage structure. This is an important requirement because the natural drainage channel is a system in relative equilibrium. This state of equilibrium exists as a result of the varying flows imposed on the channel. An alteration to this natural system caused by the drainage facility will affect the state of equilibrium of the local existing channel.

If the natural system is altered, the hydrology of the stream will be affected for the entire downstream reach and for a short distance upstream. Therefore, cross-drainage structures were designed to carry the sediment from upstream of the structure to the downstream reach as it would have moved under natural conditions.

Scour protection for cross-drainage structures was designed to withstand the highest flow velocities expected to occur. Scour protection was particularly important to the integrity of the canal siphons.

Culverts. Almost without exception, culverts (Figure 7) were designed both for open-channel flow and full pressure flow. This was necessary because of the wide range of flow conditions anticipated. At low discharges, the flow could be either subcritical or supercritical. If pressure flow with some degree of ponding were not considered acceptable, structure costs would have risen disproportionately.

Where the ponding was sufficient to have a material effect on the hydrology of the watercourse, a routing analysis of this flow through the structure was made similar to the routing of floods through a reservoir. This required development of stage-discharge relationships for the culvert and stage-storage relationships for the pond. As either inlet or outlet conditions would, at different times, control the volume of flow, the time-volume interaction of these factors also was required for design.

Overchutes. Overchutes (Figure 8) were designed as a pipe or an open flume. Pipe overchutes were hydraulically divided into four features: the inlet transition, the flume, the energy dissipator, and the outlet transition. The inlet transition usually was dependent upon whether the chute portion was designed for subcritical or supercritical flow. As with

Figure 7. Culverts

Figure 8. Overchutes

culverts, the variation in design flows usually resulted in both types of flow occurring. If the slope was available, supercritical flow produced the most economical section, and critical depth at the inlet controlled the upstream depths.

The outlet transition was generally incorporated into the energy dissipator structure. A stilling basin with chute blocks worked well for Froude numbers of three or greater. At lower Froude numbers, substitution of a baffled apron leading to the stilling basin was satisfactory. The entrance flow to the baffles or blocks was kept below critical velocity to avoid surges which would cause the water to be thrown vertically on striking the first row of blocks or jumping completely over the first or second rows. If necessary, a short additional stilling basin was incorporated just upstream of the chute entrance to ensure subcritical velocities.

Sediments

Sediments can build up to sizable amounts in canal inverts and, where feasible, were excluded from the canal. Wind-blown soils in desert and agricultural areas, particularly during cropland preparation, can produce a considerable volume of sediment.

Sediment traps were incorporated in the invert of the canal sections in certain reaches. These traps are rectangular hopper-type structures with a minimum depth of 4 feet and of variable length. Traps were installed perpendicular to the canal centerline across the full width of the invert. Some of the hoppers are uncovered, and some are divided into three sections by gratings: the first section is open for collection purposes, the second is a grizzly arrangement to prevent eddies from removing the material, and the final section is covered to retain the sediment. No mechanical equipment was designed for removal of the material. A gravity flush drain usually was not feasible; however, portable suction pumps have proven satisfactory for the limited periodic cleaning required.

Safety

The conveyance features, particularly the exposed waterways, are potential hazards to the general public and operating personnel. Safeguards to minimize the danger of hazards were standard design practice.

Fencing. Generally, protection of the public is accomplished by fencing to exclude them from extremely hazardous areas, such as check structures, siphons, pumping plant forebays, and similar dangerous areas. Otherwise, the public is allowed along the Aqueduct to fish at designated fishing access sites and reaches selected for walk-in fishing. In addition, the public has free access to designated bicycle trails (presently in the North San Joaquin and Mojave Divisions and a portion of the West Branch).

In developed and urban areas, fencing is used as protection against unauthorized trespassing and vandalism. In rural areas, fencing excludes livestock, prevents debris deposition, and defines property lines. Five-strand barbed wire generally was used for the exclusion of horses and cattle. Woven wire was used for the exclusion of smaller livestock. A 6-foot chain-link fence was used when trespass control was the main function. Six-foot chain-link fencing normally was placed around control structures and the entrances to pipelines and siphons and was extended upstream or downstream sufficiently to provide reasonable protection.

Figure 9. Canal Ladders

Ladders. Ladders for both safety (Figure 9) and maintenance purposes were provided in lined canals. The safety ladders were provided at intervals of 500 feet on alternate sides of the canal. The bottom rung of these ladders was placed a minimum distance of 3 feet vertically below the minimum normal operating water surface but at least 2 feet above invert. The top rung on the ladder was set at a maximum of 1 foot vertically below the top of the canal lining. Safety ladders were provided both upstream and downstream of transitions to checks and siphons on the secondary road side of the canal. Safety ladders also were installed downstream of concrete overchutes, bridges, fishing access points, and similar areas where people might fall into the canal. Also, safety ropes with floats span the canal and are coordinated with the ladders at these areas.

Maintenance ladders were provided upstream and downstream of transitions to checks and siphons on the primary road side of the canal and immediately downstream of the edge of concrete overchutes and bridge structures. Maintenance ladders also were provided along the canal at maximum intervals of 5,000 feet on the primary road side. These ladders extend the full length of the side slope and were anchored to the canal lining using impact-type anchors or other approved methods.

Structural Design

Reinforced Concrete. In general, concrete design, including prestressed concrete, was in accordance with Building Code Requirements for Reinforcing Concrete (ACI 318-63). The detailing procedures conformed to the Manual of Standard Practice for Detailing Reinforced Concrete Structures together with any modifications that were included in the Department's Drafting Manual.

Structural analysis and proportioning of members were based on a working stress or ultimate strength design approach. The following coefficients were used: thermal = 0.000006, shrinkage = 0.0002, and a 28-day stress of $f'_c = 3{,}000$ pounds per square inch (psi) except for concrete pipe. The allowable stresses for reinforcement were dependent upon the grade of steel used and conformed for intermediate-grade or hard billet steel to ASTM A15, for rail steel ASTM A16. Steel was deformed except in concrete pipe.

Minimum thicknesses of concrete walls were: 6 inches for single-layer reinforcement and 8 inches for walls with two layers of reinforcement, and buttress walls were a minimum of 12 inches with two layers of reinforcement. Cutoff walls were 12 inches with a depth of penetration of 2 feet, 2 feet - 6 inches, and 3 feet for water surface depths of 3 feet, 3 to 6 feet, and over 6 feet, respectively.

Structural Steel. Structural steel, with the exception of steel pressure pipe, conformed to the specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings of AISC-61. The modulus of elasticity for all grades of steel was assumed to be 29,000,000 psi and the coefficient of expansion to be 0.0000065 per degree Fahrenheit.

The following types of steel were generally specified: structural steel A36, structural rivet steel A141, high-strength bolts A325 (for structural joints), and low-carbon steel bolts A307 (for externally and internally threaded standard fasteners). The use of high-strength steel was limited to designs where its use would prove economical.

Timber and Other Materials. Basic timber design followed the requirements of National Design Specifications for Stress Grade Lumber and Its Fastenings by National Lumber Manufacturers Association of the most recent date of publication.

Various specifications on materials and methods by such organizations as the American Society of Testing Materials and the American Waterworks Association, as well as applicable federal specifications, also were used where applicable.

Railroad bridges adhered to the specifications of the most recent issue of the American Railway Engineering Association. Highway bridges conformed to the requirements of the Division of Highways or applicable county standards.

Certain loading conditions in canal structures are quite different from conventional structures. Combined earth pressures under various hydrostatic conditions often were different from those customarily encountered in design practice. Live loads and impact loads on structures were applied with judgment. Design practice, as set forth in hydraulic texts, was used as a general guide. Technical papers on different or unusual conditions often were of considerable help in providing guidance for nonstandard conditions.

No specific soil mechanics criteria or code were used due to the variation in materials encountered along the alignment of the system. Prudent design procedures were used as dictated by tests performed on the materials. Adequate testing facilities and proper interpretations of the test results were found to be essential in this phase of design.

Conveyance Features

Lined Canals

Soils and Geology. Soil and geologic characteristics were of major importance in the canal design. Soil characteristics usually determined the canal prism side slopes. The importance of having an adequate safety factor in the design of the system dictated a reasonably complete knowledge of foundation conditions through adequate soil testing.

Places suspected of having high-sulfide, soluble, reactive, or expansive materials received special study. Auxiliary studies were made where the bedrock and soil strata dipped into the canal.

Certain soils and rock, particularly some shales, exhibited considerable rebound after the overburden was removed. Although effects of this movement usually were negligible, its potential for disruption of facilities required that it be considered in the design.

The nature of the soil under the concrete lining was one of the most important aspects affecting the service of the lining. A firm, compacted, well-trimmed surface was essential. This also allowed ease of concrete placement and better assurance that the lining would perform as expected.

The danger of damage due to the removal of soluble material from the foundation with subsequent lining and/or embankment failure was not likely with most soils found in the northern part of the San Joaquin Valley. In other areas, such as in the southern part of the Valley and along the base of the foothills, soils were highly permeable and design precautions were required.

Numerous areas were encountered which contained materials that expand and contract with varying moisture content. The most common materials of this type were the various bentonite and montmorillonite clays. The thickness and location of the deposit have a definite bearing on the potential dangers. Possible differential settlement, both longitudinally and vertically, often presented greater problems than a uniform complete movement of the canal prism. The greatly reduced shear strength often associated with the expansion of clays required consideration.

Solid rock, containing little or no water but which trimmed suitably, could receive concrete lining directly. Usually, however, rock overbreak required backfill to reduce the expense of additional concrete. If backfill was used behind the lining, it was with impervious material of suitable strength characteristics. This backfill material will become saturated and must retain its moisture to prevent a buildup of hydrostatic pressure between the lining and the rock boundary and subsequent lining failure when the canal water surface is drawn down.

Prism Dimensions. The canal prism dimensions depended on the following: required waterway area, permissible side slopes, water depth-to-width ratio, freeboard, and berm size.

A side slope of $1\frac{1}{2}$:1 was used in concrete-lined canals, if slope stability requirements permitted. Steeper slopes were considered impractical but flatter slopes were used extensively.

Normal flow depth to base width ratios of approximately 0.8 in canals having a side slope of $1\frac{1}{2}$:1 were utilized since, from an economic standpoint, they were nearly as efficient hydraulically as the most efficient section which had a ratio of 1.6:1. Sections having a depth-width ratio of less than 0.5 deviate considerably from the most efficient section, both hydraulically and in required amounts of excavation and lining.

Another advantage of a high depth-width ratio was that the effect of alignment curvature on the flow was diminished. Savings in the spans of bridges, overchutes, and checks were realized by the reduced width of the section. A disadvantage of a high depth-width ratio was the greater depth fluctuation in the canal accompanying a change in discharge.

In a given canal, changes in width and depth were made in minimum increments as governed by construction trimming and lining equipment. Minimum base-width increments were 4 feet for a bottom width of 16 feet and greater, and 2 feet for lesser widths.

In fill sections, the minimum top width of compacted embankment placed was to be 6 feet. The compacted embankment sloped away from the canal at 1:1 or flatter. The remainder of the embankment was compacted to a lesser degree as its structural purpose was to support the adjacent canal operating road. Waste materials were placed outside the roadway or in designated spoil areas.

No specific relationship was prescribed between the canal water surface and the ground surface. Generally, the economics of the cost of embankment versus excavation established the normal water surface relationship to the ground surface.

Canal-bank freeboard was divided into two parts, i.e., lining freeboard and berm freeboard. Lining freeboard is the vertical distance from the design water surface of the canal to the top of the lining. Berm freeboard is the vertical distance from the top of the lining to the top of the adjacent embankment. Freeboard depended on wave action, which was dictated by the width of the water surface, or by aqueduct capacity, which was a function of operational fluctuations. In addition, the following situations affected freeboard requirements:

1. Where surges or extreme wind waves were contemplated.
2. Where design velocity was greater than 0.8 times the critical velocity. An increase of 0.5 of a foot for small canals (Q = 500 cfs or less) to 1.5 feet for large canals (Q = 2,000 cfs or more) was required depending upon the amplitude of possible surface undulations in this velocity range.
3. Where higher velocity flow occurred, such as on the outside of curves in channels, or through transition structures.
4. Where emergency shutdown conditions required upstream storage in excess of that produced by minimum freeboard values.
5. Where deep or shallow subsidence was expected to occur after construction.

A 6-foot-minimum-width berm was provided at the top of the lining in large aqueducts. The surface of this berm drained toward the canal at a maximum slope of 4:1, and the material overlapped the lip of the lining adequately to ensure drainage of the surface runoff into the canal. This material, forming the fillet on top of the berm, was placed after construction of the lining. A reduction in berm width to 4 feet was permissible for canals having a top width less than 35 feet.

Hydraulic Design. Manning's equation, although widely used, has limitation for use with large flows. These limits are affected by the nature of the flow as well as the physical parameters which must be established for each condition. It is easy to exceed the limits of Manning's equation without realizing it, although it is adequate for usual flow conditions when the hydraulic radius is about 4 to 6 feet.

For conditions beyond the limitations of Manning's equation, the Colebrook-White equation was used to modify the "n" value. This equation is based on the rational equations of Von Karmen and Prandtl for the velocity distribution in turbulent flow. The original equation was derived for flow in pipes and is the source of "f" values for use in the Darcy-Wiesbach equation for pipe flow.

Hydraulic losses, which were considered in addition to the friction loss of the moving water within the normal aqueduct prism, were due to bends, piers, or other structures which protrude into the water flow. The design gradient was checked by computing backwater curves which reflected these minor losses.

The number of piers placed in the canal flow was held to a minimum. Prestressed design of bridges had increased economical spans to lengths which could either clear the water prism or require only one pier. Pipelines and utility crossings often were attached to or passed through vehicle bridges.

Losses of head in bends were minimized by using as large a bend radius as possible, maintaining a minimum bend radius of 5 times the canal top width avoiding "s" curves, and keeping structure piers out of bends where practicable.

A minimum average velocity of 3 feet per second at design flows was adopted to provide flushing action during cleaning and maintenance activities. The design velocities utilized were between 3 and 4 feet per second and were based on economical comparisons of conduit size versus pumping costs.

Pipelines

Selection. Pipelines for the present facilities were used extensively only for the North Bay Aqueduct, the South Bay Aqueduct, and the Santa Ana Division of the California Aqueduct. Pipelines operating under pressure are suited for the conveyance of water in terrain where rapid changes in elevation occur. The criteria for penstocks and discharge lines are discussed in Volume IV of this bulletin. Tunnels, which are a form of cast-in-place pipelines, are discussed later in this chapter. Design details of pipelines are more fully covered in other chapters of this volume but are briefly discussed here as appurtenances to canal reaches.

The use of steel or concrete pipe generally was an economic choice with both alternatives meeting the hydraulic requirements of the Project. In many cases, availability of the various types of pipe and the bids of pipe manufacturers were the determining factors in the total cost of a contract. Therefore, unless some special condition clearly required specifying the type of pipe to be used, the design and specifications allowed a choice of material by the bidders.

Steel Pipe. One advantage which steel pipe has over concrete is that bedding or backfill for steel pipe is not as critical as for concrete pipe.

The principal allowable stresses for steel pipe are:

Type	*Designation (ASTM)*	*Yield Stress (psi)*	*Ultimate Strength (psi)*	*Design Stress (psi)*
Structural	A283-C	30,000	50,000	15,000
Carbon Steel	A285-C	30,000	50,000	15,000
	A441			
High Strength	¾ of an inch and less	50,000	70,000	25,000
Low Alloy	¾ of an inch to 1½ inches	46,000	67,000	23,000
	1½ inches to 4 inches	42,000	63,000	21,000

The required mortar lining thickness was designed with a minimum compressive stress of 4,500 psi in either case. Enamel coating or protective wrappings followed AWWA standards. Design for abnormal transient pressure, such as power failure at a pumping plant, used 1.5 normal design stress. Where negative pressure occurred, a safety factor of 1.33 against collapsing was used. Earth loads were, in general, calculated using Marston's formula or a suitable derivative, with deflection based on Spangler's formula.

Concrete Pipe. Concrete-pipe specifications called for four general types: cast-in-place reinforced pipe; shop-formed reinforced pipe, either cylinder or noncylinder; and precast prestressed pipe. With the exception of cast-in-place pipe, specified sizes conformed to those diameters generally available from pipe fabricators. Bedding for concrete pipe was a critical factor, and a margin of safety was provided as compensation for possible construction irregularities. A minimum bedding angle of 90 degrees was required.

Cast-in-place pipe was used for heads up to 125 feet and at locations where precast pipe was not available or was uneconomical. A minimum cover of 3 feet was adopted. A minimum shell thickness of 8 inches was provided to allow for two layers of steel, with not less than 40% of the steel in the outer layer. Watertightness was important, and conduit joints were spaced at approximately 25 feet using 9-inch waterstops without joint fillers.

Vertical and horizontal curves were designed for thrust forces. The radius of such curves was established so that the resultant force on the pipe (neglecting weight of backfill material) would not be inclined greater than 10 degrees from the vertical. Minimum radii for convex vertical curves were set so that the weight component of the pipe plus water was at least 50% greater than the upward component due to water pressure.

Precast noncylinder pipe was used for heads between 50 and 125 feet, with precast cylinder pipe for higher heads. Noncylinder pipe was of the two-cage reinforcement, vertically or centrifugally spun, lock-joint, rubber-gasket type; cylinder pipe was the same except it was vertically cast.

To obtain the 90-degree bedding, backfill was compacted for a vertical distance not less than 25% of the outside diameter of the pipe. In rock or rocky ground, proper bedding was assured by overexcavation and replacement with compacted material.

Hoop tension for cylinder pipe was designed for 50% of the yield strength of the cylinder. For hoop tension combined with earth load, the ultimate strength method was used with a 1.8 load factor. This assumed the center of gravity will occur between the inner and outer layers with the resultant force applied at the center of gravity. Bedding and angle change requirements were the same for cylinder pipe as for noncylinder pipe.

Appurtenances. Pipelines require distinctive operational devices. Manholes were provided for access at approximately 2,000-foot intervals and specifically at all summits and low points in the pipeline. They were buried where periodic access was not expected to be necessary. Air valves, vents, or blowoffs often were incorporated with the manholes.

Combination air release vacuum valves were used at high points of a pipeline to release air during pipeline filling, or to provide air when the pipeline was emptied. Air release-vacuum valves were sized for an air intake equivalent to the discharge from all blowoffs operating simultaneously. The minimum-size valve used was 4 inches for concrete and 6 inches for steel pipe. The air valves usually were isolated from the pipe by the use of rising-stem gate valves.

Blowoffs were provided to drain lines for operational or emergency reasons. Experience with operation of the South Bay Aqueduct indicated 6-inch plug valves were the most dependable. When the blowoff would be operable before the static head reduced below 75 feet, an energy dissipator also was required. The jet stream from the blowoff with even relatively low heads could be objectionable and require containment.

Corrosion and Cathodic Protection. One or both of two methods were used to protect steel pipelines from corrosion, that is, prevent or minimize the corrosion process or the tendency of the pipeline to act as the cathode in the electrolytic process. The first method was to coat the interior and/or exterior of the pipe with inert substances. The second method was to provide a substitute material or anode which would in itself become subject to the corrosion process rather than the pipeline.

Two types of interior coatings were used: coal-tar or cement mortar. Cement mortar has a longer life and is still effective after slight cracking because of its alkaline nature. Coal-tar has a lower initial cost and is more flexible but has a shorter life. Cement mortar usually was used for a single-barrel pipeline where substitute service could not be provided as with a multibarrel line. The lining was either shop- or field-applied. However, if field-applied, a thicker lining was required. Interior coatings possessed the added advantage of improving the flow characteristics of the pipeline.

The exterior coating, which was field-applied, consisted of a coating of hot coal-tar onto which asbestos felt was impressed, followed by another coal-tar coat and a layer of kraft paper.

Cathodic protection was provided by using sacrificial or impressed anodes. The sacrificial anodes usually were magnesium or zinc, which are naturally reactive to iron or steel, and no outside source of current was supplied. The impressed anodes usually were carbon or graphite, and the driving force or current was supplied through a rectifier.

Corrosion test stations were established at approximately 500- to 1,000-foot intervals to monitor corrosion and cathodic protection.

Sections of steel pipeline embedded in concrete were exterior-coated whether the entire pipeline was embedded or not. All mechanical couplings of the noninsulating type were bonded to assure electrical continuity of adjacent sections. Reinforced-concrete pipe sections were bonded together by thermite welding a jumper cable to the steel rings at the bell and spigot connection.

Tunnels

Tunnels, as conveyance features of the State Water Project, perform as pipelines or closed conduits flowing under pressure. The basic difference from pipelines is that rather than being buried underground or supported aboveground under relatively determinable loading conditions, the tunnel penetrates rock and soil masses which load the tunnel lining with forces that are not subject to finite determination.

In view of the uncertainties as to rock behavior, a complex structural analysis leading to design criteria was not warranted. However, tunnels for the Project were designed with definite concepts, such as: (1) the tunnel would be concrete-lined, (2) the lining would not be reinforced, and (3) the tunnel would be pressure-grouted after the lining was in place. It was assumed that after grouting the lining and tunnel rock were in contact with each other. Additionally, the following factors were considered in tunnel design: (1) the shape of the driven tunnel need not conform to the shape of the finished section; (2) any support system within the tunnel for use during tunnel construction would not, even where incorporated into the tunnel lining, contribute to resisting the stresses in the lining; and (3) the rock walls will, like backfill on a pipeline, contribute to the resistance of the internal pressure in the tunnel.

As the internal pressure in the tunnel is supported by a combination of the resistive stresses of the unreinforced lining and the rock, the required lining thickness is dependent on the modulus of deformation of the confining rock. The depth of rock required to dissipate stress as a function of the tunnel radius was assumed to be 2.5 times the radius.

Concrete lining after placement will shrink and, on subsequent filling of the conduit, will expand through absorption of moisture. Tests with prestressed concrete pipe have indicated that expansion after filling is approximately equal to the shrinkage occurring during the early hardening of the concrete. The exact amount of expansion which will occur in an unreinforced tunnel lining is not known but was assumed to be 75% of the shrinkage coefficient. Therefore, based on the assumption that tunnel grouting will occur after shrinkage of the lining and will result in negligible voids between the lining and the rock, the rock must support this expansion stress.

Tunnels, like pipelines, were provided with inspection manholes and the protection of air valves and blowoffs. These could be provided only at or adjacent to the tunnel portals, because the expense of underground surge chambers or adit connections prohibited their incorporation in general tunnel design. Surge chambers in tunnels downstream from pumping plants or upstream of power plants are an exception.

Construction

Construction contracts for conveyance facilities of the State Water Project were awarded and administered in accordance with provisions of the State Contract Act, Sections 14250 to 14424, Government Code, Statutes of the State of California. The State Contract Act requires that bids be solicited in writing and that the contract be awarded to the lowest responsible bidder. To comply with the Act, the following procedures were employed.

1. Prequalification of Prospective Contractors—A two-phase prequalification procedure was used to establish qualified bidder lists of those contractors desiring to bid. First, if the required financial statement indicated that the contractor had the necessary resources, the request for prequalification was processed further. Second, the contractor's ability was assessed based on the firm's overall experience and other uniform factors.

2. Advertisement and Award of Contracts—Public notice of a project was given once a week for at least two consecutive weeks in a newspaper published in the county in which the project was located and in a trade paper of general circulation in either San Francisco or Los Angeles as appropriate. A "Notice to Contractors", in each case, was sent to all contractors on the list of qualified bidders. This document generally described the requirements and extent of the work and indicated the time and place for receiving the bids.

Contracts were awarded to the lowest responsible bidder. Responsible bids were those meeting all the conditions of bidding stated in the bidding requirements and determined to be reasonable in cost when compared with the engineer's estimate.

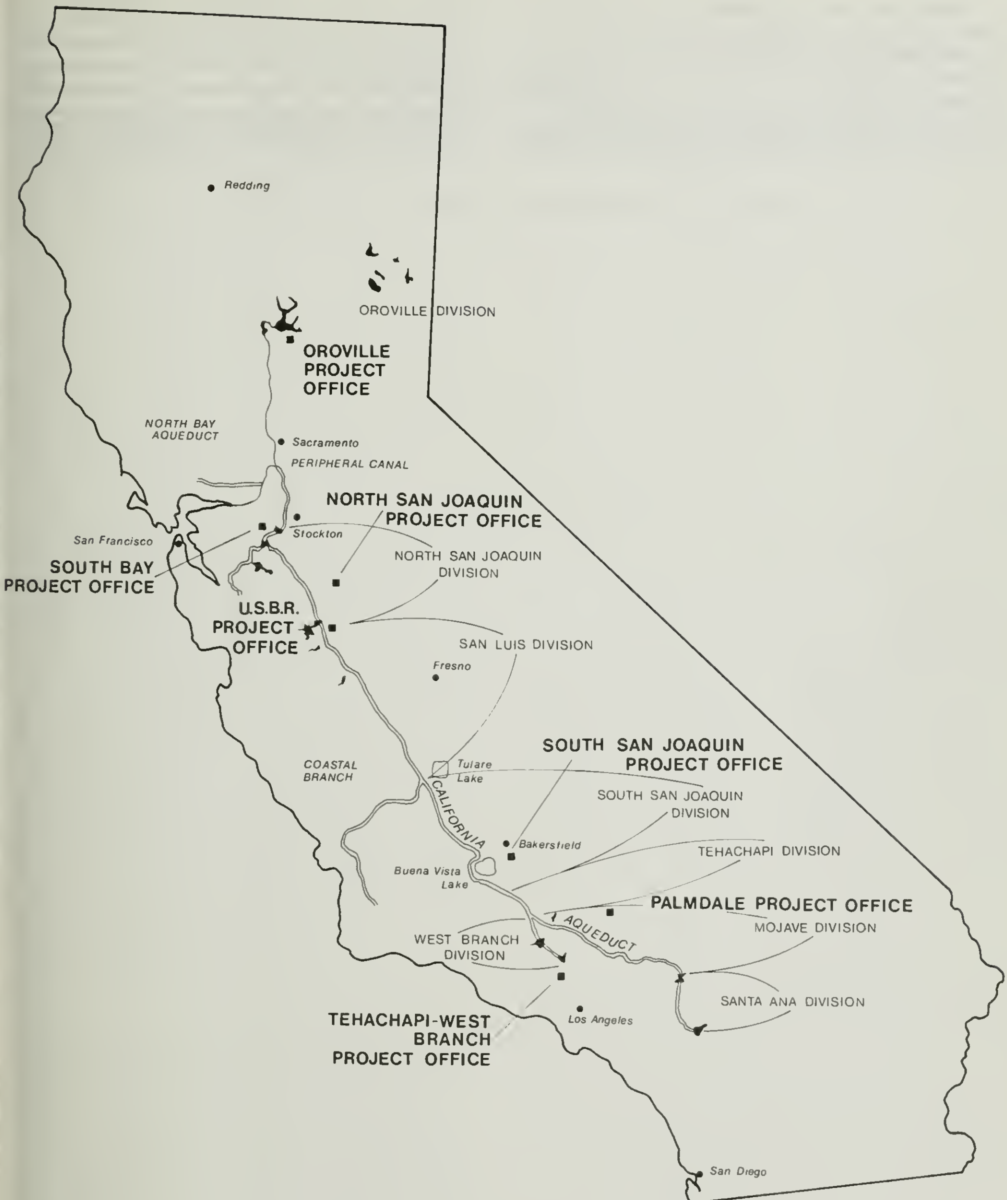

Figure 10. Location of Construction Project Offices

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The Department's organization for supervision of construction activities consisted of project offices at selected locations throughout the State and a headquarters construction office located in Sacramento (Figure 10). Each project office was responsible for all project construction work within a particular geographical area and was staffed with construction engineers, inspectors, engineering geologists, and laboratory and and other technicians. The headquarters construction office provided administrative and liaison services to the project offices. Factory inspection of materials and equipment to be incorporated in the work was performed by an equipment and materials section of the headquarters construction office.

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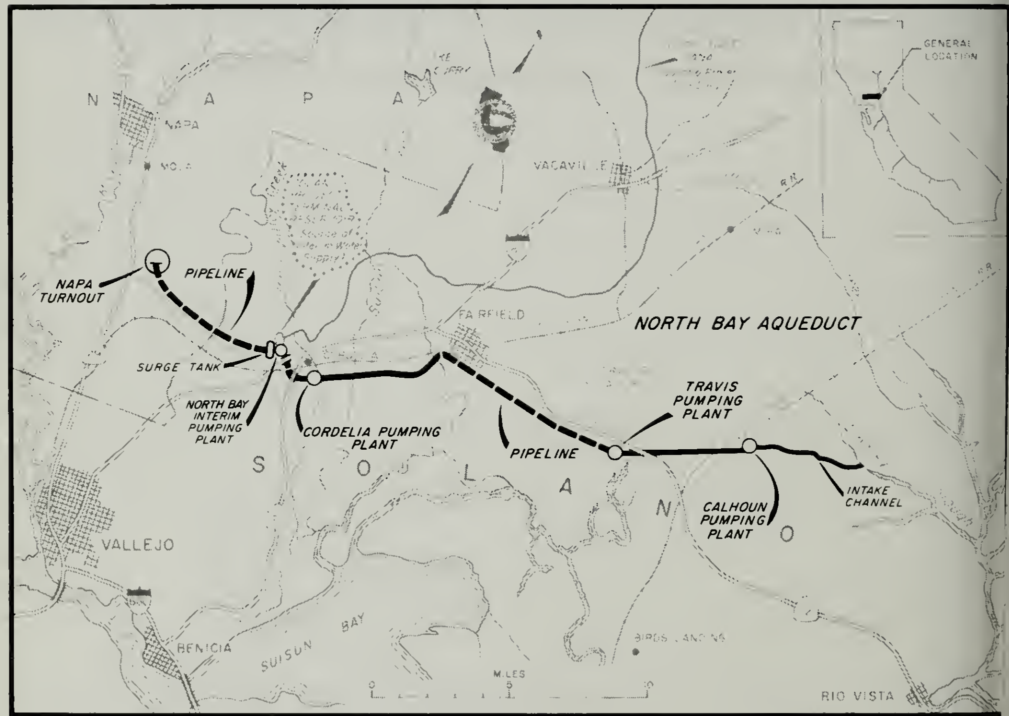

Figure 11. Location Map—North Bay Aqueduct

CHAPTER II. NORTH BAY AQUEDUCT

Introduction

Role in the State Water Project

North Bay Aqueduct, located in Solano and eastern Napa Counties (Figure 11), is a water conveyance system to supply water for municipal use in the cities of Vacaville, Fairfield, Suisun City, Benicia, and Vallejo in Solano County and to the city of Napa and the American Canyon Water District in Napa County.

Phase I of the Aqueduct delivers water to the Napa County area by means of a temporary supply from the terminal reservoir of the U.S. Bureau of Reclamation's Solano Project, which obtains its water from Lake Berryessa via the Putah South Canal. Phase II will connect Phase I to a water supply from the Sacramento–San Joaquin Delta and also will serve the various Solano County cities previously mentioned.

History

Engineering feasibility and economic studies of North Bay Aqueduct in 1957 visualized a major aqueduct, tunnel, and reservoir complete with pumping plants that would supply Delta water to three counties: Marin, Napa, and Solano. A reduction in the estimated needs for water in this service area eliminated Marin County from the service area, and the original design objective was revised to encompass a much smaller service area in Solano and eastern Napa Counties. The revision also included consideration that a portion of Solano County's needs for the City of Vallejo would be met directly from the Delta and transported through Vallejo's own facilities.

The Department of Water Resources' studies during the years 1957–1959 were directed toward reanalysis of previously studied aqueduct routes. These studies resulted in the adoption of a plan to construct the project in two phases: Phase I, obtaining an interim supply of water from the Solano Project, and Phase II, supplying all the water from Calhoon Cut/Lindsey Slough which is located on the western edge of the Delta.

Geography, Topography, and Climate

The aqueduct route starts in the western Delta waterways off Cache Slough, passes through gently contoured grazing land east of Travis Air Force Base, skirts the northern edge of Suisun Marsh, and goes through the rolling grass-covered hills west of Cordelia by way of Jameson Canyon to its terminal on the east side of the Napa River Valley. Only minor streams draining into Suisun Marsh are crossed by the aqueduct route. Major transportation arteries intersected by the aqueduct route from east to west are: State Highway 113, Sacramento Northern Railroad, State Highway 12, Cordelia Road, State Highway 21, Southern Pacific Company Railroad, and Interstate 80 and State Highway 12 twice again. Communities along the route are Fairfield, Suisun City, and Cordelia. Elevation of the aqueduct water surface will range from low tide at the Delta to about 450 feet where the Aqueduct passes over a hill just west of Cordelia.

The climate of the North Bay area can be classed as mild with an annual average temperature of 60 degrees. The prevailing west-southwest winds (with a mean hourly speed of 10 to 15 mph) tend to temper the climate. Due to the location in relation to Sacramento Valley, summer temperatures can reach 100 degrees Fahrenheit, and winter temperatures are lower than those around San Francisco Bay, a few miles away. Average annual rainfall at Fairfield is nearly 24 inches.

Present Facilities

Phase I of the North Bay Aqueduct consists of interim and permanent facilities which supply water to the Napa County Flood Control and Water Conservation District.

This phase, which uses excess yield from the Solano Project, is expected to be available only up to 1980. The District contracts directly with Solano County for the water which the Department delivers through the present facilities. After 1980, no excess yield will be available under present estimates. Interim facilities are: intake piping from Solano Terminal Reservoir, an outdoor pumping plant, and a discharge line up to Cordelia Surge Tank. Permanent facilities constructed under Phase I are the Cordelia Surge Tank, Napa Pipeline, Creston Surge Tank, and Napa Turnout Reservoir.

Future Facilities

Phase II facilities will use most of the Phase I facilities, except the North Bay Interim Pumping Plant and discharge line up to Cordelia Surge Tank, and are planned presently to be completed in 1980 to meet water-user demands in Napa and Solano Counties until 2024.

Phase II facilities will serve the Napa County Flood Control and Water Conservation District and the Solano County Flood Control and Water Conservation District, whose maximum annual entitlements by about 1990 will reach 25,000 acre-feet and 42,000 acre-feet, respectively.

Phase II facilities, as presently conceived, will consist of the Calhoun Pumping Plant to be built on Calhoun Cut approximately 19 miles to the east of the present interim plant, an open-channel and pipeline

Figure 12. Cordelia Discharge Line and Surge Tank

conveyance system to the present Cordelia Surge Tank, a booster pumping plant (Travis) halfway along the line, and a final pumping plant (Cordelia).

Features

In this chapter, only Napa Pipeline, Cordelia Surge Tank, and Napa Turnout Reservoir will be covered (Figure 12). The Interim Pumping Plant and discharge line to Cordelia Surge Tank are covered in Volume IV of this bulletin. Statistical summaries of North Bay Aqueduct conveyance facilities and terminal storage facility are presented in Tables 1 and 2.

The Solano Project Terminal Reservoir is a pond formed by an earth embankment. At one end of the embankment is a reinforced-concrete inlet structure from which water is divided for the City of Benicia's pumping plant, the City of Vallejo's Monticello Pumping Plant, and the Department's Interim Pumping Plant.

The Interim Pumping Plant's four electric-motor pumping units lift a maximum flow of 23 cubic feet per second (cfs), 319.5 feet through the 24-inch discharge line to Cordelia Surge Tank, which is approximately one-half mile away at elevation 445.7 feet. From Cordelia Surge Tank, water flows by gravity

TABLE 2. Statistical Summary of Storage Facility—North Bay Aqueduct

NAPA TERMINAL RESERVOIR

Type
Steel tank

Data

Diameter	190 feet
Height	35 feet
Capacity	5,000,000 gallons

Inlet-Outlet
Terminus of 36-inch, concrete, cylinder pipe from North Bay Aqueduct—tank rides on aqueduct

Outlets
36-inch Napa turnout—control by water user; outflow, 72-inch riser inside tank—top of riser at elevation 353.8 feet—open, 18-inch, reinforced-concrete, overflow line to Fagan Creek; drain, 8-inch welded-steel pipe controlled by 8-inch butterfly valve empties into 18-inch overflow pipe

through a buried 4.2-mile, 36-inch, pretensioned-concrete cylinder pipe to the 5,000,000-gallon Napa Turnout Reservoir terminus on a hill adjacent to Napa's water treatment plant.

Creston Surge Tank is located on a rise in Jameson Canyon, through which the pipeline passes, approximately 2 miles west of Cordelia Surge Tank.

Geology and Soils

The reach from Cordelia Surge Tank to Napa Turnout Reservoir involves four lithologic units of earth and rock:

1. Recent Landslide Deposits—These deposits consist of lean clay and fine sandy clay. They are present along approximately 4% of the route.
2. Slope Wash, Earth Flow, and Ancient Landslide Deposits—These consist of light clay and usually include some scattered, angular, sandstone blocks; present along approximately 20% of the route, mostly overlying the Markley formation.
3. Alluvium—This consists of heavy-to-light clay with some localized sand and gravel deposits near the stream channels; present along 65% of the route. Alluvium generally was not suitable for construction purposes.
4. Markley Formation—Consists mostly of friable silty sandstone and shale; present along approximately 11% of the route.

TABLE 1. Statistical Summary of North Bay Aqueduct

Aqueduct Reach	Type of Conveyance or Facility	Inside Diameter (Inches)	Capacity (Cubic feet per second)	Length (Miles)
Temporary Discharge Line	Steel pipeline	24	23	0.5
Permanent Discharge Line	Concrete cylinder pipeline	36	46	4.2
Terminal Reservoir	Steel Tank	Data not applicable—See Table 2		

OPERATIONS
Manual on-site control or remote shutoff from area control center, Delta Field Division.

Design

Napa Pipeline

This reach consists of 4.2 miles of steel pipeline and appurtenant structures. The Pipeline traverses hilly terrain and is buried 3 to 10 feet underground.

The alignment of the Aqueduct generally was determined by the topography in the area. The Aqueduct necessarily passes through Jameson Canyon; any other routing would have required a tunnel or a higher pumping head to lift the water over the range of hills between the Cordelia area and Napa Valley.

Pipeline. The pipeline size was determined by considering a range of pipe diameters versus the cost of the system. A present-worth cost comparison included the cost of the installation, pumping cost, head-loss cost, and other cost factors and showed that a 36-inch-diameter pipe would be the most economical means of conveyance.

One important consideration in design of the Pipeline was the decision to assume greater construction costs to lessen right-of-way acquisition costs. The Pipeline was placed at sufficient depth and cover so that future developments, such as subdivision, roads, sewer lines, and water mains, could be installed with minimum disruption of the Pipeline. In areas where subdivisions were a definite possibility, the Pipeline was required to have at least 5 feet of cover, including approximately 2½ feet of consolidated or compacted material.

Three alternative types of pipe were considered in the pipeline design, and bidders were given a competitive choice. They were: (1) steel pipe, (2) pretensioned-concrete cylinder pipe, and (3) asbestos cement pipe. The contractor chose alternative (2).

The design of pretensioned-concrete cylinder pipe was based on American Pipe and Construction Company data, utilizing a 1.5 deflection lag factor (Figure 13).

Minimum design thickness of the steel cylinder was No. 10 gauge; joints were welded. Deflection was controlled by the allowable width of a crack in the pipe coating. In essence, the pipe was designed on the basis of a 10% probability of a crack 0.01 of an inch wide.

The lining and coating, as designed, were assumed to provide strength in combination with the steel cylinder for support of the external loads. Internal pressure did not govern design.

Alignment. Alignment for the Pipeline was controlled by two fixed points. One point was the terminal reservoir site set by joint agreement with Napa County Flood Control and Water Conservation District, for which the elevation and an approximate location were given. The other point, which was referred to as "Hill 343" because of its elevation, was the site for Cordelia Surge Tank at the beginning of the line. This hill is visible on Figure 12.

Within 2 miles of Napa Turnout Reservoir, all alignment alternatives were identical, the major constraint being the maintenance of a set distance of 20 feet from other pipelines running through the area. At a point near the county line between Solano and Napa Counties, available right of way was quite narrow, so the Pipeline was located halfway between existing pipelines and railroad or other facilities.

Right of Way. The pipeline right of way was obtained by both permanent easements and "fee simple" purchase and was 60 feet wide, with 40 feet on one side of the Pipeline and 20 feet on the other. The 40 feet was necessary for the maintenance of an access road.

This layout was used for most of the Pipeline, except that the 40/20 ratio was adjusted in extremely narrow locations. In addition, a temporary 20-foot construction easement on both sides of the Pipeline was specified.

Earthwork. Open-cut excavation, which was used at the bench just before the first Highway 12 crossing (Figure 14), was conducted along the face of the hill to provide a working area during construction and a permanent operations and maintenance roadway along the Pipeline. There also was open-cut excavation at the Cordelia Surge Tank site.

Trench excavation was divided into three categories: Category 1 included areas generally considered to be normal excavation, Category 2 encompassed the bench-cut area and was expected to be a sandstone material, and Category 3 included areas of high ground water.

Backfill material for the Pipeline consisted of consolidated backfill, compacted backfill, and loose backfill. Some compaction was applied at the surface of the trench.

Consolidated backfill was a fine-grained cohesionless material placed to the top of the pipe on slopes of less than 30%. This material was jetted and vibrated to obtain a minimum relative density of 70%.

Compacted backfill was obtained from the trench excavation as often as possible. This material was required to be compacted to the top of the pipe to 95% compaction. In areas of potential future development, at least 2½ feet of compacted backfill was placed over the pipe.

Loose backfill was placed over consolidated or compacted backfill as necessary to bring pipeline cover to grade.

Vents. Vent structures are located at two points on Napa Pipeline. They provide economical and fail-safe means of supplying air to high points of the Pipeline, thus preventing a vacuum in the pipe in the event there is an abrupt stoppage of flow and water drains away from the high points. Vent No. 1, located about 3,600 feet from Cordelia Surge Tank, consists of a 6-inch-diameter steel pipe that is attached to Napa Pipeline and is buried up a cut slope for some 83 feet and then is free-standing for about 20 feet above original ground. This type of structure, rather than an air

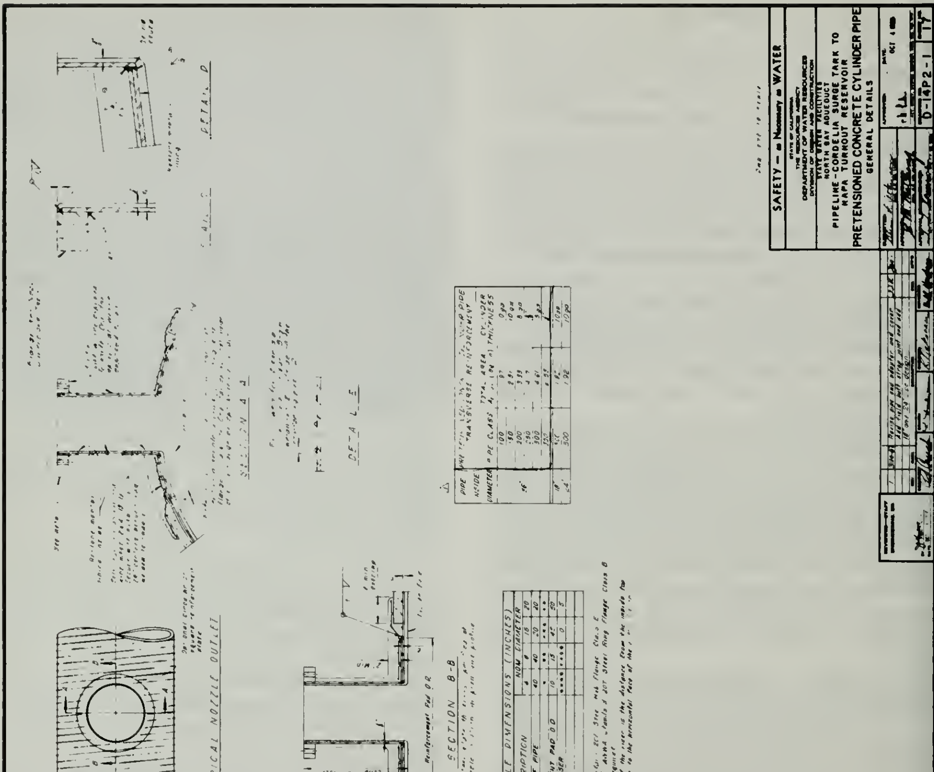

Figure 13. Pretensioned-Concrete Cylinder Pipe Details

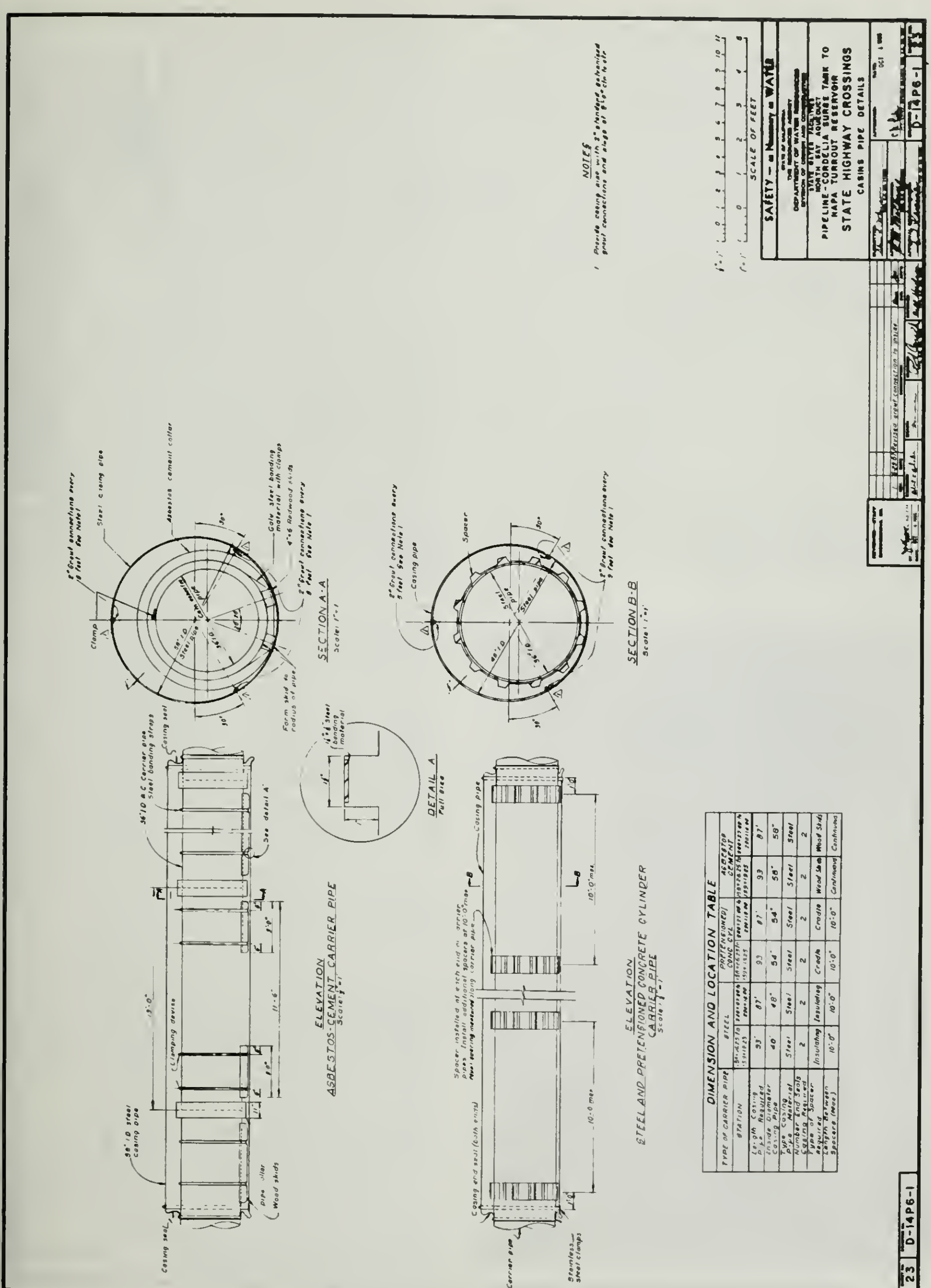

Figure 14. Pipeline Crossing—State Highway 12

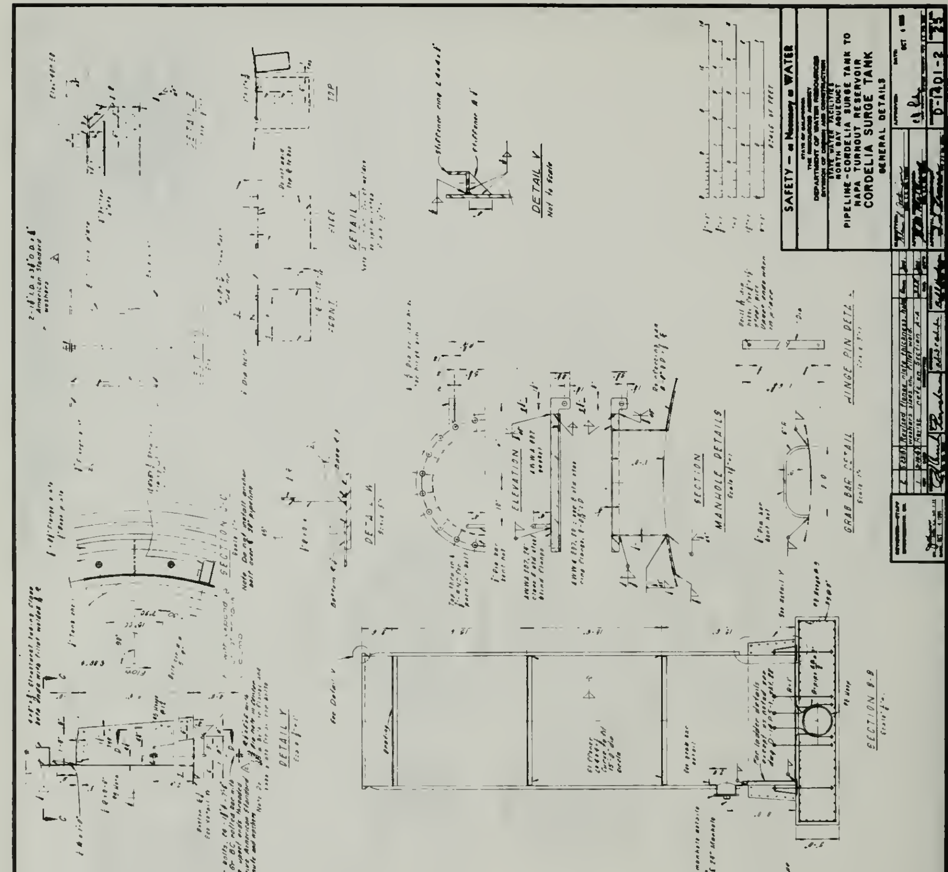

Figure 15. Cordelia Surge Tank Details

valve, was installed at this particular location because the elevation of the Pipeline approaches the hydraulic gradeline at lower flows, and experience on other pipelines has shown that air valves are not completely reliable in such a situation.

Vent No. 2, located about 4,300 feet from Cordelia Surge Tank, is an open-top, 35-inch-diameter, 42-foot-high, steel standpipe joined to the top of Napa Pipeline without any valves. This vent has the capability of readily supplying the large volume of air that would be required in the case of the Pipeline being suddenly emptied downstream of the vent location.

Surge Tanks. Cordelia Surge Tank (Figure 15) provides (1) an open water surface to maintain the maximum hydraulic gradeline, (2) the volume of water necessary to maintain a full discharge line during the valve closure period, and (3) a dampening effect for the water-hammer pressures. The tank is 44.5 feet high by 15 feet in diameter and is located approximately 2,500 feet from the Interim Pumping Plant. It rests on a concrete foundation 18 feet in diameter and 5 feet deep through which the Pipeline passes (Figure 16).

Creston Surge Tank is a one-way surge tank located on a point approximately 2 miles west of Cordelia Surge Tank and is intended to protect the Pipeline from water column separation in the portion of the Pipeline between Vent No. 2 and Napa Turnout Resevoir. The tank is 10 feet high by 20 feet in diameter and is connected to the Pipeline by approximately 37 feet of 24-inch line. A valving system allows pipeline flow under high pressure to bypass the tank.

American Canyon Turnout. Just inside the Napa County line, a 36-inch tee was incorporated to serve as a future turnout for the American Canyon Water District. The tee was capped with a standard blind flange.

Appurtenances. Vents provided at two high points along the Pipeline admit air into it when operating conditions lower the hydraulic gradeline to the point of pipeline profile intersection.

Blowoffs to facilitate draining of low points of the line are valved nozzles connected to the Pipeline. Each blowoff includes a 6-inch nozzle and a 4-inch valve and extension pipe.

Automatic combination air release-vacuum valves were installed at high points in the line to release large amounts of air at the time of pipeline filling and to remove small amounts of entrained air when the Pipeline is in operation. All manholes have an inside diameter of 20 inches and are not more than 1,500 feet apart.

Chain-link fencing surrounds the Interim Pumping Plant and plant switchyard (grounded to conform to Pacific Gas and Electric Company regulations).

Cathodic Protection. The cathodic protection system for Napa Pipeline consists of two deep-well anodes, cast magnesium anodes in the soil at Creston Surge Tank, and extruded magnesium anodes inside Creston Surge Tank. In addition, impressed cathodic protection with corrosion test stations were installed at intervals of about 500 feet along the Pipeline, with cathodic interference test stations at foreign pipeline crossings. Dielectric connections electrically insulate the Pipeline and reservoir from the City of Napa facilities.

Monitoring stations have been included to make possible a check on corrosion and cathodic protection.

Napa Turnout Reservoir

Napa Turnout Reservoir (Figures 17, 18, and 19) is a cylindrical, steel, open-top tank approximately 190 feet in diameter and 35 feet high which is used to store water delivered to Napa County Flood Control and Water Conservation District. It is the terminal structure of Napa Pipeline.

Sizing of the Reservoir involved a comparison of various reservoir combinations of depths and diameters. Pumping costs as well as tank costs were included in the comparison.

Storage in the Reservoir consists of dead storage, the 5,000,000-gallon storage requested by Napa County Flood Control and Water Conservation District, a mismatch storage, and emergency storage. When the top of the emergency storage is reached, water will spill over a riser and be dumped into Fagan Creek.

Figure 16. Cordelia Surge Tank

Figure 17. Interior of Napa Turnout Reservoir

The limiting elevation for dead storage was calculated to be approximately 326 feet. This figure was computed using the hydraulic gradeline controlling factor elevation of 325 feet plus the exit losses and velocity head necessary to get maximum design (46 cfs) flow through the reservoir outlet pipe.

Mismatch storage, which compensates for any difference between the pumping flow rates and the demand rates, was determined by using the magnitude of incremental changes in flow rate that the pumps are capable of making. Emergency reservoir storage was sufficient (619,400 gallons) to provide pipeline delivery at the maximum design flow rate for at least one-half hour. An allowance of 1 or 2 feet of freeboard brought the total reservoir height to 35 feet.

The Reservoir was required to be located in the vicinity of Napa's water treatment plant. The location was further dictated by the sloping topography in the treatment plant vicinity and by a requirement that water be delivered at a hydraulic gradeline of not less than elevation 325 feet. A further site consideration was to minimize reservoir bowl excavation.

Three soil borings were analyzed, and it was decided that to minimize differential settlement over materials with differing in-place densities, the entire tank bottom should be placed on undisturbed native soil. After excavation to subgrade, the subgrade was prepared by disking or harrowing, with thorough removal of all projecting rock to preclude any punching effect through the reservoir bottom. Soil, either in situ or imported, was compacted at least 12 inches below grade. A 3-inch layer of oiled sand was placed on top of the compacted backfill to provide a reservoir cushion.

Manually operated butterfly valves were provided on both reservoir inlet and outlet lines to prevent uncontrolled drainage of the Reservoir in the event of a pipeline failure.

The reservoir shell was fabricated of ASTM Designation A131 steel. The type of steel was determined by considering various strengths versus material costs. The reservoir shell bottom slopes one-half of 1% outboard from the center of the Reservoir to ensure complete dewatering of the Reservoir and to minimize puddling. The inlet and outlet pipelines were fabricated of ASTM Designation A283, Grade C steel.

The centerline of the outlet pipe was placed 2 feet - 9 inches above the base plate of the Reservoir to prevent the entrace of dirt and algae into the pipe. The centerline of the outlet pipe was located at elevation 323.25 feet, which resulted in a constant depth of submergence of approximately one-half the diameter of the outlet pipe, considered sufficient to prevent pipe vortex action. A 1-foot-wide reinforcing collar was welded around the outlet pipe on each side of the shell.

The overflow oulet riser for the Reservoir is a 72-inch reinforced-concrete pipe. Water overflowing into this pipe will be carried away by an 18-inch, welded-steel, overflow, drain line. The pipe centerline is approximately 20 inches above the reservoir floor.

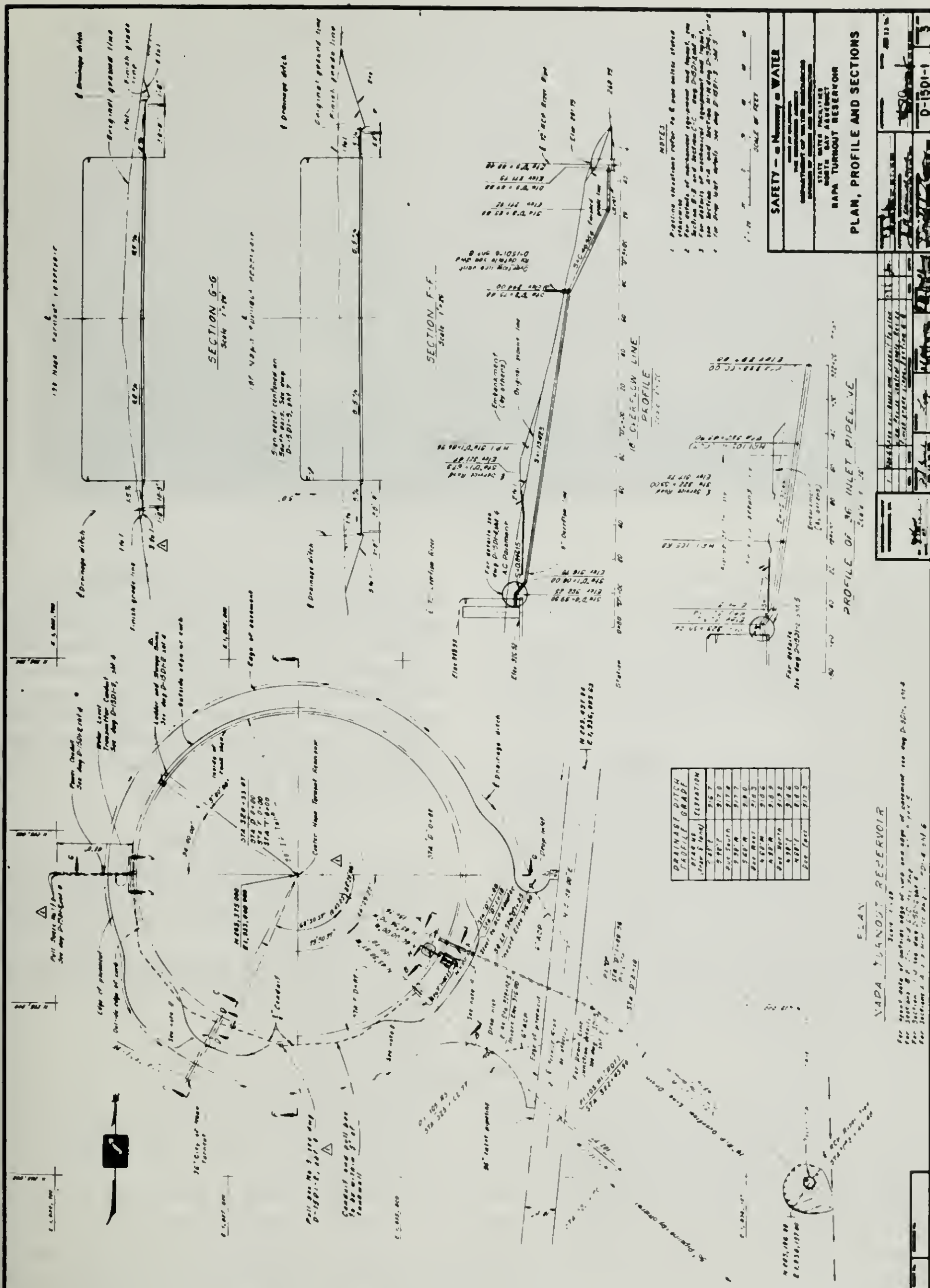

Figure 18. Napa Turnout Reservoir—Plan, Profile, and Sections

Figure 19. Completed Napa Turnout Reservoir

An 8-inch, welded-steel, drain pipe buried under the Reservoir is used to completely drain the Reservoir for inspection, maintenance, and repairs. This pipe leads to a reinforced-concrete valve box containing an 8-inch butterfly valve and a steel coupling. From this valve box, the 8-inch pipe runs to a steel connection with the 18-inch, steel, reservoir, overflow line (Figure 20).

The steel, overflow, drain line outside the reservoir wall is supported by a concrete anchor. Two sleeve couplings are suspended between the reservoir wall and the anchor to provide articulation in case of differential movement. From this anchor, the overflow drain line extends underground for approximately 10 feet and then transitions into the 18-inch reinforced-concrete pipe that discharges into Fagan Creek.

An asphalt-concrete access road approximately 15 feet wide was provided completely around the Reservoir which permits access and egress of a standard highway truck from either direction. A concrete curb ring is provided around the Reservoir to prevent vehicles from bumping into the Reservoir or piping. The sloping roadway drains outward from the Reservoir to a ditch which carries the water to drop inlets.

A ladder extending inside and outside the Reservoir and a climbing pole provide access and egress. The ladder has a safety harness to clasp onto the pole.

Napa Turnout Reservoir was not fenced because of the proximity of Napa's water treatment plant; security is furnished by the City of Napa.

Construction

Construction supervision was administered by the South Bay Project Office located in Livermore, California. A trailer office for inspectors was located at the construction site.

General information about the three major contracts for the construction of Phase I of the facilities is shown in Table 3.

TABLE 3. Major Contracts—North Bay Aqueduct

	Specification	Low bid amount	Final contract cost	Total cost—change orders	Starting date	Completion date	Prime contractor
Solano Reservoir to Cordelia Surge Tank	67-25	$224,437	$276,212	$24,444	8/31/67	1/ 7/69	Cabildo Corporation
Cordelia Surge Tank to Napa Reservoir	67-05	998,689	942,745	2,202	2/ 7/67	1/ 4/68	Lentz Construction Co., Inc.
Napa Turnout Reservoir	67-21	271,362	276,593	2,663	5/25/67	4/ 5/68	A. Teichert & Son, Inc.

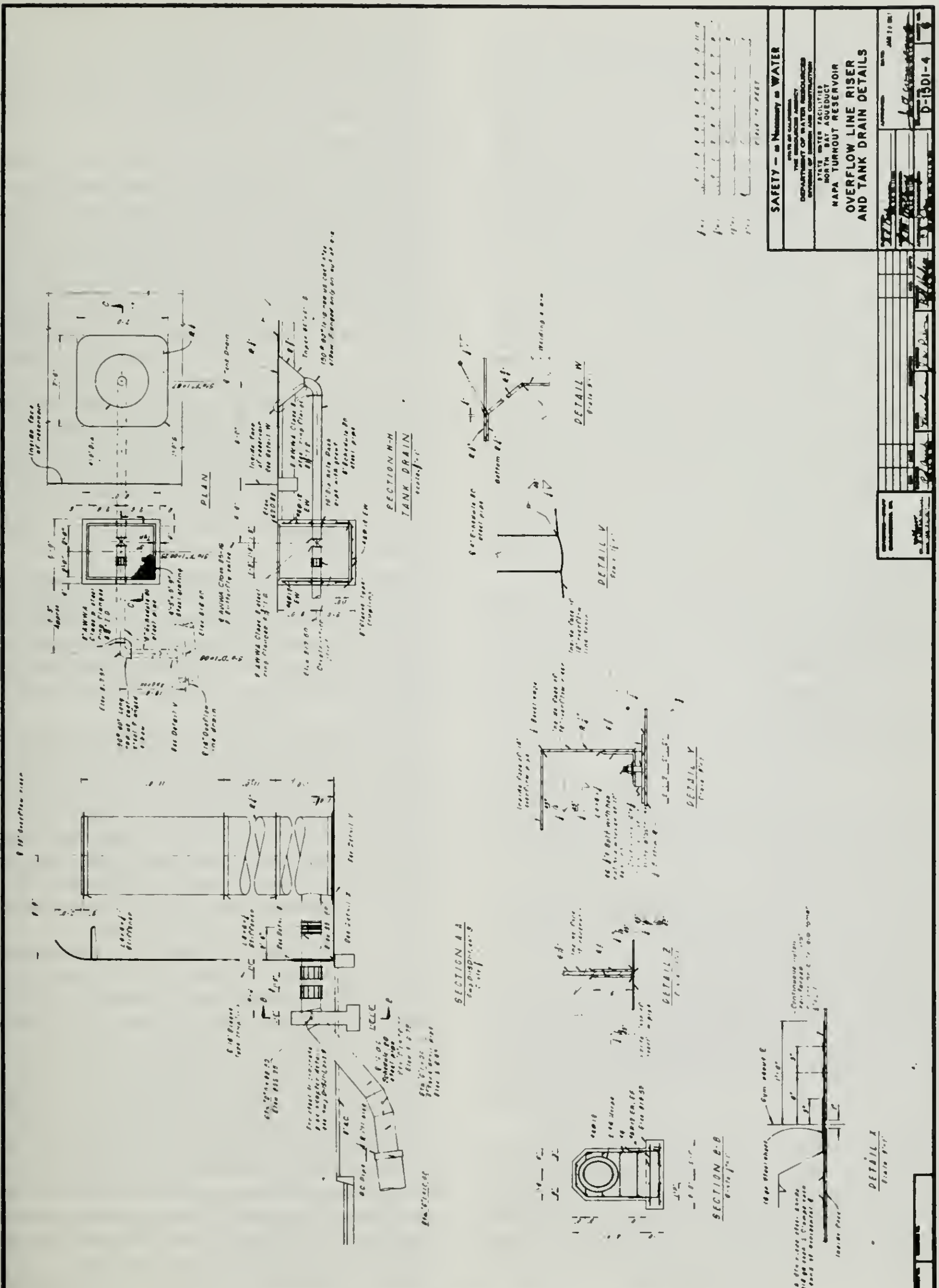

Figure 20. Overflow Line Riser and Tank Drain Details

Cordelia Surge Tank to Napa Turnout Reservoir

Cordelia Surge Tank. Cordelia Surge Tank consists of a ¼-inch-thick steel tank anchored to a thick concrete foundation with a 36-inch pipeline running through the base. As water is pumping through the Pipeline under normal conditions of pressure, it rises through a nozzle into the tank to approximately one-half of the height of the tank and is free to rise and fall with fluctuations in the pressure, thus dissipating the amount of surge going through the Pipeline. The top of the tank is open to the air except for a platform of steel grating which is 3 feet below the top.

Erection of Cordelia Surge Tank began on September 18, 1967. The procedure was to assemble the ½-inch base ring; tack-weld the bottom plates to the ring and to the flange on the 36-inch-diameter nozzle; assemble the three sections of a ring; and weld the vertical joints on the inside, back-gouge, and then weld the outside. After welding the vertical joints, the next higher ring was erected. After this ring was tack-welded into position, the lowest horizontal joint was welded in the same manner as the vertical joints. There usually was an additional ring atop those that were being welded; this provided greater ease in matching joints and also permitted work inside the tank at the same time. The crew completed the erection of the base and shell work on September 28, 1967.

On October 10, 1967, erection of Cordelia Surge Tank was completed and, on October 16, 1967, preparations began for the protective coating. First, the surface was sandblasted inside and out. Then, it was coated with inorganic zinc silicate (Diametcote #3) and was allowed additional time to cure because of the cold damp weather. The final colored coating of vinyl paint then was applied by spraying.

The final coat of blue with the stiffener ribs painted black gives a pleasing appearance to the tank, which can be seen from Interstate Highway 80 to the east and northeast and State Highway 21 to the south. It is a distinctive landmark.

Pipeline Earthwork. Clearing and grubbing were minimal, with the removal of only a few trees in three small crossings. Common excavation was required in two open-cut areas to lower the ground surface to a level that would afford reasonable trench depth. Some of this material had to be ripped. Later, a large portion was used for compacted backfill. Pipe trench excavation was by a self-propelled trenching machine assembled from parts of various machines. The cross section trenched out by this machine had a bottom width of 5 feet, vertical sides up to about 3 feet, and ¾:1 slopes to ground surface.

This machine could not operate on sharp curves, in hard material, in wet ground, or where side clearance was restricted. These problem areas were skippped by the trencher to be excavated by other means. On steep hills (over 20%), it was assisted by a large tractor used to pull it up the hill and to act as a brake to prevent the trencher from rolling backward.

Most miscellaneous excavation was done by a small backhoe. Pipe trench in problem areas and wet spots was excavated by backhoe draglines and a truck crane with a clamshell bucket. "Bellholes" under pipe joints were dug by a backhoe. Bedding material was dumped into the trench by front-end loaders, then spread to grade by hand.

Material excavated from the trench was pushed into the trench as backfill after all pipe installation was completed. The only compaction done was by the tractor pushing in the material after cover over the pipe reached 3 feet. Ground surface was restored as nearly as possible to the original level. Backfill at 95% relative compaction was placed in specified locations to restore county roads, private driveways, creek crossings, and as plugs at intervals along the Pipeline to prevent ground water from percolating along the pipe.

Compaction about the pipe was done with hand-held gasoline-powered tampers. In larger areas, a small tractor towed a vibrating roller over the fill. Consolidated backfill (⅜-inch sand) was placed in three lifts to a depth of 2½ feet above the top of the pipe, each lift being jetted to saturation with water and then vibrated with hand-held gasoline-powered tampers.

Pipe. Approximately 22,200 feet (4.2 miles) of 36-inch, modified, pretensioned-concrete cylinder pipe was furnished in several classes in designated reaches. For certain reaches, it was overcoated with coal-tar epoxy to minimize corrosion by electrolytic action between the Pipeline and adjacent pipes owned by others.

The modified, pretensioned-concrete cylinder pipe consists of a steel cylinder which was formed by shaping and welding together long strips (coils) of steel of specified type and thickness in a continuous process machine, which produced a helical seam and cut the cylinder into 40-foot or other lengths. The cylinder was subjected to a hydrostatic test before being coated with mortar, wrapped with reinforcing steel rods at a predetermined stress, coated with a dense covering of cement mortar, and stored for curing.

The pipe was manufactured at a plant in Livermore beginning on May 29, 1967. The pipe sections were hauled to the job site on truck trailers equipped with padded cradles to fit the curvature of the pipe and were unloaded as close as practicable to the station where each piece was scheduled to be placed. Each piece had been marked at the plant with a number for identification. Prior to unloading the pipe, piles of earth had been pushed up by a bulldozer to form cradles for the pipe to facilitate handling. Plastic covers had been wired on both ends of the pipe at the plant to prevent drying, but most of these blew off in transit and had to be replaced if the pipe was not placed in the trench immediately. Later, plywood covers were used to retain the plastic.

Pipe laying began on August 25, 1967 near the downstream end of the line. The pipe was designed so that the spigot was placed upstream and the bell of the next pipe section fit over it. Each length of pipe was lifted into place in the prepared trench by a "side-boom" cat crane and fitted to the previous section by two pipefitters, then tack-welded to hold it in place. In some locations, a truck crane was used to lay the pipe. Certain short stretches were skipped in the initial pipe-laying operation to bypass critical work areas.

Field welding was performed by the shielded-metal arc method. One welder usually stayed with the pipe-laying crew to tack-weld while other welders followed behind to make the complete welds. As each welder completed a joint, he marked it with his initials for identification by the Department's inspector. Welding was inspected visually and spot-checked by radiographic means.

After welding, the joints were coated with mortar inside and outside. Access to the inside was through manholes which were closed with a bolted flange and coated with mortar before being covered with backfill.

When the Pipeline was considered installed and operational, it was subjected to hydrostatic tests for water tightness. Approved testing procedures were as follows:

Test No. 1—A bulkhead was welded to the pipe at the joint just upstream from Vent No. 2. The line was filled with water from an adjacent main of the Vallejo Water Department. Water surface elevation was determined by measuring down from the grating floor in the top of the tank. At the end of the 24-hour test period, the water had dropped 0.01 of a foot in the tank. This was a loss of 14 gallons in 0.82 of a mile, 17% of the allowable 100 gallons per mile. This test was completed December 9, 1967.

Test No. 2—After Vent No. 2 was installed, all of the Pipeline was filled with water to approximate elevation 390 feet, which was just below the base of Cordelia Surge Tank. This meant that a portion was tested twice. Measurement was along the sloping inside of the pipe. Loss at the end of the 24-hour test period was 53% of allowable. This test was completed December 27, 1967.

Procedures, materials, and equipment for radiographing special fittings and field welds in the steel pipe conformed to the requirements of ASME Boiler and Pressure-Vessel Code (Section VIII, paragraph UW-51). This was done in addition to the normal requirements of AWWA Standard D-100.

Vent No. 1 of the Pipeline consists primarily of a 6-inch-diameter, Schedule 40, steel pipe mortar-lined and coated, approximately 105 feet along with a 12-inch-radius bend at the 36-inch pipe. Approximately 24 feet of this length has an exterior coating of inorganic zinc silicate. This is the free-standing portion which is exposed to the air above the encasement of concrete. The erection crew consisted of one pipefitter, one welder, and one small rubber-tired backhoe with operator.

Vent No. 2 consists basically of a horizontal, 14-foot-long by 36-inch-inside-diameter, steel pipe with a 36-inch-inside-diameter nozzle at midpoint. Welded to the nozzle is a 42-foot-high length of 36-inch-inside-diameter, ¼-inch-thick, steel pipe. The curved horizontal section and a portion of the vertical section are encased in an 11 by 11 by 6-foot structural concrete base.

The 42-foot vertical section is exposed above the structure concrete encasement approximately 38 feet and has an exterior coating of inorganic zinc silicate. The interior coating from the nozzle to the top of the vent is coal-tar epoxy, Type C-200.

The interior of the nozzle and 14-foot-long horizontal section is mortar-lined. Before the sleeve-type couplings were buried, they were encased in coal-tar enamel. The 14-foot horizontal section was placed with a large side-boom tractor, leveled, and secured before erecting the free-standing section.

The free-standing section was erected with a 35-ton truck crane and secured in a plumb position with three temporary guy wires attached to buried timbers acting as deadmen, then welded. After all work was completed, the guy wires were removed.

It was necessary to jack the pipe casing under State Highway 12 at two locations. Pits were dug on both sides of the Highway, just outside the right-of-way fence, to receive the tracks which supported the casing before it was pushed into the excavation by a hydraulic jack. The pit allowed room for welding together the sections of pipe before inserting them into the casing.

Excavation of the bore was done by hand-held pneumatic tools. Material was removed from the casing by a wheelbarrow and from the pit by a truck crane with clamshell.

Stakes for alignment and grade were set in the pit. These were carefully observed on the first crossing, and the 36-inch pipe was inserted without incident. On the second crossing, the boring crew could not hold the casing to proper alignment and grade. When halfway through, a check showed it to be off grade by more than 1 foot and off line ½ foot. It was decided to start anew on the opposite side of the Highway. This was done so the two halves met reasonably well but resulted in an angle point in the casing which made it impossible to support the pipe on skids. It was decided to fill the entire casing around the 36-inch pipe with grout. This gave firm support and eliminated the difficulty of coating the joints with mortar in the normal manner.

Asphaltic Concrete. Asphaltic concrete was used to pave approaches from Highway 12 to the two access roads. This included preparing the roadbeds of the access roads, placing and compacting the subbase and base material, and placing the asphaltic concrete.

Creston Surge Tank. This tank provides a reserve supply of water that automatically fills the 36-inch line in the event negative pressures develop. The tank stands 10 feet high above ground, is 20 feet in diameter, and was fabricated from structural steel ¼-inch thick. Two 18-inch-diameter swing check valves, adjusted to a differential head of 2 feet with the low-head side of the valves toward the tank, prevent outflow of water from the tank into the 36-inch line except in an emergency. A 3-inch line bypasses the swing check valves and maintains the required water level in the tank by allowing water from the 36-inch line to flow into it. The automatic control on the 3-inch line is a 3-inch valve with hydromercury control. Actual erection of the tank began on November 17, 1967, after placement and curing of the concrete foundation had been completed.

The erection procedure was: (1) assemble the roof on the ground, (2) assemble the bottom outside ring that forms part of both the floor and the wall shell of the tank and tack-weld the floor plates to the ring, (3) erect and weld the shell, (4) weld the floor plates, (5) lift the roof assembly into place and weld to the shell, and (6) weld the 4- by 8-inch stiffeners to the shell. Butt welds were back-gouged with a carbon-air arc before the backside weld was made. Welding was inspected visually and by radiographic means for conformance with Appendix C of AWWA Standard D-100.

Cathodic Protection. This included a complete cathodic-protection system, discussed fully in an earlier section of this chapter. A major part of the work was drilling two 450-foot-deep wells for the deep-well anodes, installing the anodes in the wells, and backfilling with cokebreeze and gravel.

Structural Concrete. Structural concrete was placed in the following features: base of Cordelia Surge Tank, pipe encasement and anchor block encasement of blowoffs and air valves, encasement of the 36-inch turnout for American Canyon Water District, base of Creston Surge Tank, encasement of the twin 20-inch nozzles for Creston Surge Tank, walls and floor of the valve vault, and encasements at the bases of two pipeline vents. A total of 351 cubic yards of concrete was supplied by a ready-mix plant south of Napa.

Figure 21. Tank Site Excavation

Figure 22. Oiled Sand Pad for Tank

The concrete mix was in the proportions of five sacks of Type V cement, 70 pounds of pozzolan, and 0.2 pounds of pozzolan 8 water-reducing agent. Curing was by a pigmented sealing compound. Forms for the concrete structure were made from plywood. To keep labor costs to a minimum, the contractor avoided building forms wherever possible by placing concrete in the full width of the trench and using sandbag dikes in place of end forms.

Miscellaneous Concrete. Miscellaneous concrete was used under the base of Cordelia Surge Tank, under the base of Vent No. 1, and for fence-post anchorage. As solid rock was not encountered under the Surge Tank, the invert of the foundation was overexcavated 5 feet and backfilled with nonreinforced concrete. Design mix was the same as structural concrete.

Reinforcing Steel. Reinforcing bars were fabricated both in the shop and in the field. A total of 15,500 pounds of reinforcement was used.

Napa Turnout Reservoir

Excavation. Field construction work on Napa Turnout Reservoir started with open-cut excavation on June 12, 1967. Removal of the overburden was made by two scraper units and a tractor. Compaction

Figure 23. Welding Tank Floor Plates

of the subbase was obtained by a double-drum sheepsfoot roller pulled by a crawler tractor. Pipe trenches were dug by a 2-cubic-yard backhoe, a small backhoe, and a combination backhoe-loader tractor. Tight sandstone near the bottom in the outlet pipe trench stopped the combination tractor and was removed with a jackhammer and pick-and-shovel work (Figure 21).

Tank Foundation. A 1-foot-thick layer of selected cohesive material compacted to 95% density was laid down as a subbase with the same equipment that compacted the natural ground foundation base. A 3-inch-thick base mat composed of a mixture of sand and oil was placed on top of the compacted cohesive soil base. Approximately 260 cubic yards of this sand and oil mixture containing $1\frac{1}{2}$ gallons of oil per cubic foot of sand was placed and rolled to grade by August 8, 1967 (Figure 21).

Structural Concrete. Structural concrete included: the tank ring, the curb ring, inlet and outlet pipe anchor blocks, and a valve vault. The 1-foot-thick by 1.5-foot-wide, 95-foot-radius, tank ring was placed in one day (August 2, 1967). A five-sack mix with $1\frac{1}{2}$-inch maximum size aggregate was used to make the 42-cubic-yard placement. Concrete placement was completed by December 26, 1968. All structural concrete was cured with white pigmented compound.

Tank Erection. Erection of the steel-plate tank was started on August 21, 1967 by welding rim plates to the wall footing around the tank ring (Figure 22). Next, the floor plates were laid out and welded. Vertical seam welds of the tank wall were welded by hand, whereas horizontal seams were machine-welded. All welding of the tank, including the 72-inch overflow riser, was completed by October 20, 1967 (Figures 23 and 24).

Tank Painting. All interior surfaces were sandblasted to bright metal and the welds ground smooth. The interior surface coating of the tank was composed of four 1.5-mil coats of VR-3 vinyl paint. The primary coat was brush-applied, and the three final coats were sprayed on. Three colors were used: the first coat was black, the second and third coats gray, and the final coat white (Figure 25). The floor received the same color sequence as the walls.

The exterior walls were sandblasted to white metal; then a 2.5-mil primary coat of inorganic zinc silicate was applied and the entire surface spray-cured with a catalyst.

The exterior appearance of the painted tank was not pleasing because of a streaked gray surface. However, this condition was eliminated after complete curing by natural weather. The final color, with the paint completely cured, is a grayish blue.

Peripheral Roadway. On January 22, 1968, the contractor started rough grading of the base of the roadway. Because of the limited working area around the tank, a self-loading scraper (paddlewheel) was used to remove the surplus dirt. Fine grading was done with a grader. The subgrade was completed on January 26, 1968, but heavy rains prevented placement of base course aggregate until March 4, 1968. On March 7, the road base was prime-coated with liquid asphalt, and the final paving with asphaltic concrete plant mix was completed. The seal coat was applied on March 13, and a final coating of sand and oil was applied on March 27, 1968.

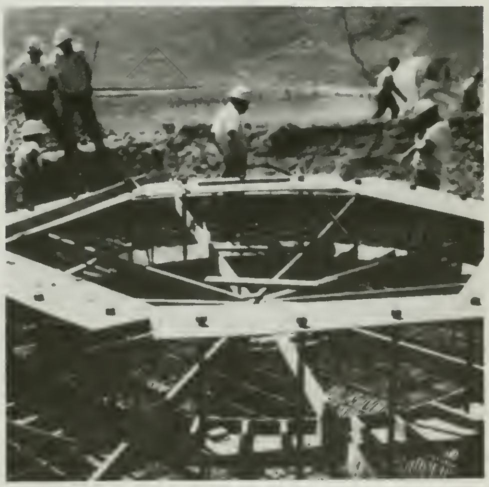

Figure 24. Tank Construction

Figure 25. Hand Painting Inside of Tank

A farm-type tractor with a screed attachment spread the plant-mixed asphalt, and compaction was achieved with a steel-drum roller. The area between the tank ring and the curb ring was paved on March 6, 1968. On March 26, 1968, water from North Bay Interim Pumping Plant started to fill the Reservoir. The 33-foot level in the Reservoir (the top of the overflow riser) was reached on March 30, 1968. The intake valve then was closed, and a 24-hour observation of the system at full operating head was started. The test complied with all requirements of AWWA procedures. The first water delivery was made from the Reservoir to the City of Napa on April 5, 1968.

Electrical Installation. Electrical work consisted of installation of conduit and electrical boxes, placement of cast magnesium anodes, installation of jumper cables on the sleeve-type coupling, conduit around the Reservoir to the inlet valve, and an additional pull box and terminal box.

Installation of the four magnesium anodes, the insulating test station, and necessary wiring was completed on February 12, 1968.

Jumper cables, to ensure complete conductivity between the pipe shell and flexible coupling, were attached by thermite welding. The welds were coated with hot coal-tar enamel and painted with inorganic zinc silicate.

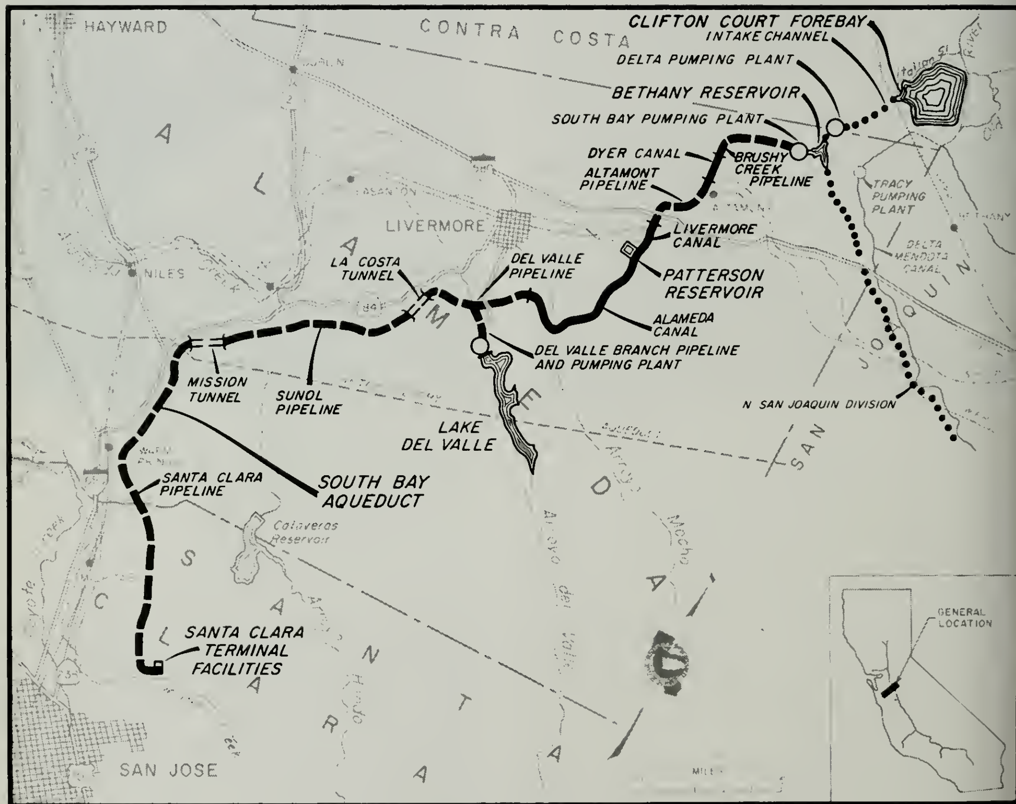

Figure 26. Location Map—South Bay Aqueduct

CHAPTER III. SOUTH BAY AQUEDUCT

Introduction

Role in the State Water Project

South Bay Aqueduct was the first delivery system completed in the State Water Project. Design and construction extended over an 11-year period, from June 1958 to July 1969. Thus, this aqueduct antedated the California Aqueduct and the Delta Pumping Plant.

South Bay Aqueduct serves portions of Alameda and Santa Clara Counties with a maximum, scheduled, annual delivery of 188,000 acre-feet and has the capacity for an additional service of about 22,000 acre-feet annually.

Hydraulic Function

The 44.7-mile system consists of 11 miles of canal, 31.8 miles of pipeline, and 1.9 miles of tunnel. South Bay Pumping Plant has sufficient pump lift to provide gravity flow through the balance of the system (Figure 26).

Regulation and storage were provided about one-third the distance along the system by Del Valle Dam, Lake Del Valle, and Del Valle Pumping Plant. The Aqueduct terminates in a large steel tank located in east San Jose.

Geography, Topography, and Climate

Service areas are located in the Livermore, Santa Clara, and adjacent valleys near San Francisco Bay. The Diablo Range, trending northwest-southeast, separates Santa Clara Valley from the San Joaquin Valley to the east. Livermore Valley is the largest valley in the Diablo Range and is an important route for highways and railroads. Santa Clara Valley is one of the most historic valleys in Central California.

The climate, tempered by its closeness to the ocean and San Francisco Bay, is semiarid with warm-to-hot summers and cool moist winters. During construction, extreme temperatures ranged from 18 to 112 degrees Fahrenheit. Precipitation usually occurs only during the winter months. Average annual rainfall ranges from about 11 inches at South Bay Pumping Plant to about 15 inches at San Jose in the Santa Clara Valley.

The area is changing rapidly from an agricultural and rural environment to urban communities.

Features

The Aqueduct originates at Bethany Reservoir, an enlarged section of the California Aqueduct 1 mile downstream from Delta Pumping Plant. Prior to completion of Delta Pumping Plant on the California Aqueduct, an interim pumping plant and a 2-mile, unlined, interim canal connecting to the U.S. Bureau of Reclamation's Delta-Mendota Canal transported water to Bethany Reservoir.

At Bethany Reservoir, South Bay Pumping Plant lifts aqueduct water through a total design head of 611 feet up the eastern ridge of the Diablo Range. Each of the two pump discharge lines terminates in surge tanks located on the ridge crest. The conveyance facilities beginning at these tanks consist of two parallel Brushy Creek pipelines, which extend for a distance of 2.4 miles to the Dyer back-surge pool. This pool empties into the concrete-lined Dyer Canal, which for 1.9 miles conveys water along the western ridge of the divide between the San Joaquin and Livermore Valleys. Dyer Canal terminates in Altamont Canyon at a transition into the 2.3-mile, single-barrel, Altamont Pipeline. This pipeline continues along the Canyon and turns south in a tunnel under U.S. Highway 50 to the head of Livermore Valley Canal. The

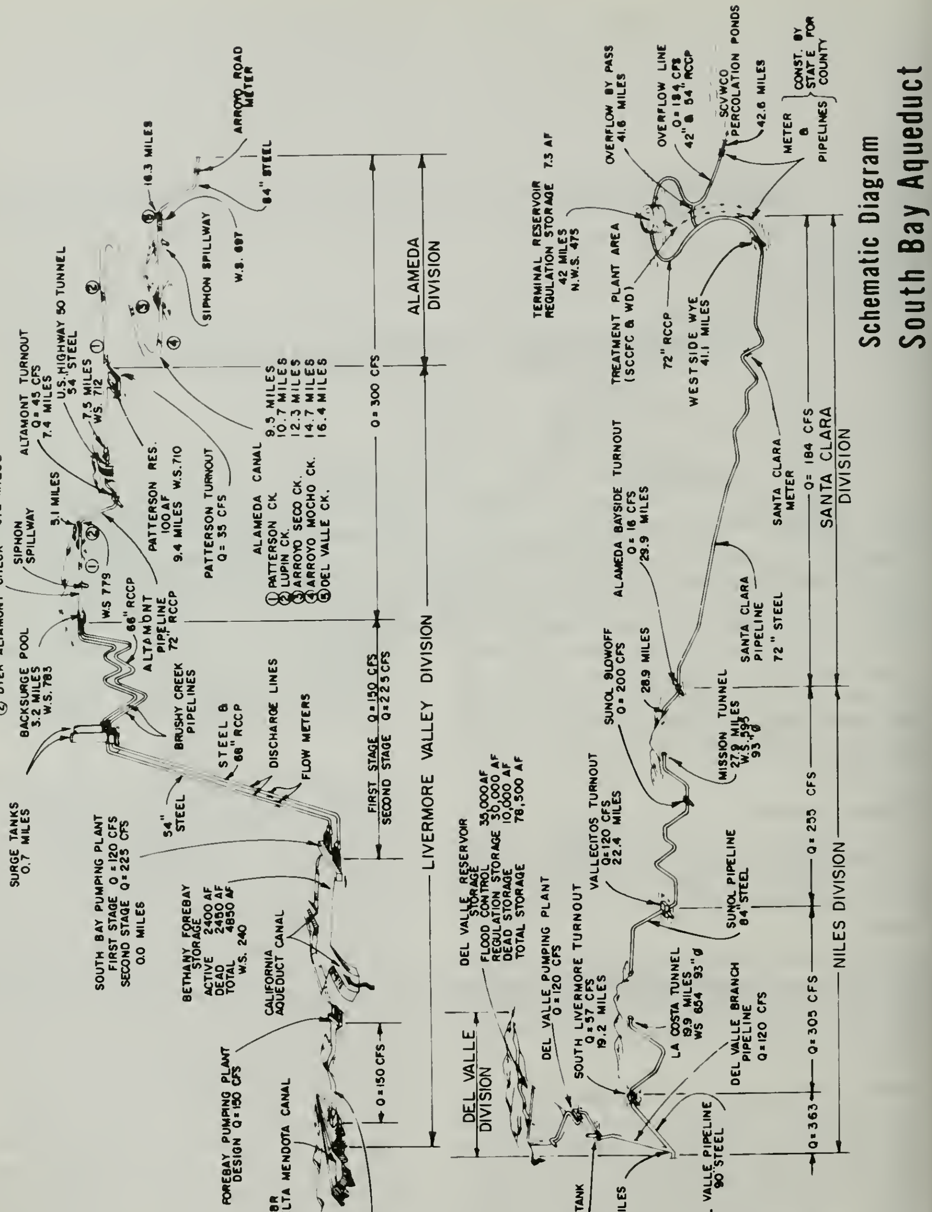

Figure 27. Schematic Profile of South Bay Aqueduct

2.2-mile-long, concrete-lined, Livermore Valley Canal terminates at Patterson Reservoir. This small (100-acre-foot) diked reservoir provides off-line storage for the Alameda County Flood Control and Water Conservation District's Zone 7 Treatment Plant. The canal section continues for another 6.9 miles as the Alameda Canal.

At this point, the Aqueduct again becomes a pressure conveyance system. Del Valle Pipeline continues along Livermore Valley for 3.6 miles, crosses the mouth of Arroyo Del Valle, and terminates at La Costa Tunnel.

Approximately 2 miles up this arroyo, Del Valle Pumping Plant, Del Valle Dam, and Lake Del Valle provide off-line regulation and storage for the system through Del Valle Branch Pipeline.

The Aqueduct crosses into La Costa Valley through the 1-mile La Costa Tunnel and continues across Sunol Valley through the 6.9-mile Sunol Pipeline before entering Santa Clara Valley through the 0.7-mile Mission Tunnel. The Aqueduct then is formed by the 14.2-mile Santa Clara Pipeline which terminates northeast of the City of San Jose at the terminal facilities. These facilities include the 0.9-mile-long terminal pipeline, a steel-tank terminal reservoir, and a 0.6-mile-long overflow pipeline which spills into Santa Clara County's Penitencia Creek percolation ponds (Figure 27).

Turnouts from Altamont Pipeline, Livermore Valley Canal, Alameda Canal, and Del Valle and Sunol Pipelines serve Alameda County Flood Control and Water Conservation District, Zone 7. The Alameda Bayside turnout at the beginning of Santa Clara Pipeline serves Alameda County Water District in the Fremont area. Turnouts from Santa Clara Pipeline and terminal facilities serve Santa Clara Valley Water District.

The conveyance features are discussed later in this chapter. Details of the South Bay Pumping Plant, interim pumping plant and canal, and the Del Valle Pumping Plant and discharge lines are included in Volume IV; Del Valle Dam in Volume III; and South Bay Aqueduct control system facilities in Volume V, all of this bulletin. Statistical summaries of South Bay Aqueduct conveyance facilities and certain storage facilities are presented in Tables 4 and 5.

TABLE 4. Statistical Summary of South Bay Aqueduct

Aqueduct Reach	Type of Conveyance or Facility	Inside Diameter (Inches)	Capacity (Cubic feet per second)	Length (Miles)
Brushy Creek				
No. 1	Reinforced-concrete pipeline	54	120	2.4
No. 2	Concrete cylinder pipeline	66	225	2.4
Dyer	Canal, concrete-lined—trapezoidal—checked	*	300	1.9
Altamont	Reinforced-concrete pipeline	72	300	2.3
Highway 50	Steel pipe in 93-inch-diameter casing tunnel	54	300	0.2
Livermore	Canal, concrete-lined—trapezoidal—checked	*	300	2.2
Patterson	Asphalt-lined reservoir	Data not applicable—See Table 5		
Alameda	Canal, concrete-lined—trapezoidal—checked	*	300	6.9
Del Valle	Steel pipeline	84	300	2.2
	Steel pipeline	90	363 to 305	1.4
La Costa	Concrete-lined horseshoe tunnel finished to circular shape	93	305	1.0
Sunol	Steel pipeline	90	305	0.1
	Steel pipeline	84	305 to 255	6.7
	Steel pipeline	90	255	0.1
Mission	Concrete-lined horseshoe tunnel finished to circular shape	93	255	0.7
Niles	Steel pipeline	90	255	0.1
	Steel pipeline	72	255	0.4
Santa Clara	Steel pipeline	72	184	12.8
Terminal	Reinforced-concrete pipeline	72	184	0.9
Terminal Reservoir	Steel tank	Data not applicable—See Table 5		
Terminal Overflow	Reinforced-concrete pipeline	42	184	0.4
	Reinforced-concrete pipeline	54	184	0.2
Del Valle	Concrete cylinder pipeline	60	120	1.5

OPERATIONS
Manual on-site control or remote control from area control center, South Bay Field Division.

* Canal data: Bottom width, 8 feet; depth of lining, 6 (Dyer only) and 7 feet; depth of flow, 4.67 (Dyer only) and 5.58 feet; side slopes, 1½:1; lining thickness, 2½ to 3½ inches; check structure, 5 one-radial-gate structures and 2 two-radial-gate structures

TABLE 5. Statistical Summary of Certain Storage Facilities—South Bay Aqueduct

PATTERSON RESERVOIR

Type
Compacted embankment dam

Lining
3 inches of open graded asphalt over clay blanket

Data		
Crest elevation	712.5	feet
Crest width	15	feet
Crest length	1,275	feet
Structural height above foundation	33	feet
Freeboard, maximum operating surface	2.5	feet
Maximum operating storage	100	acre-feet
Maximum operating surface elevation	709.63	feet

Inlet		
175-foot weir on side of Livermore Canal		
Capacity	300	cubic feet per second
Elevation of weir	708.82	feet

Outlet
42-inch-diameter pipeline to Alameda Flood Control and Water Conservation District, Zone 7 treatment plant—releases regulated by water user

SANTA CLARA TERMINAL RESERVOIR

Type
Steel Tank

Data		
Diameter	160	feet
Height	20	feet
Capacity	2,500,000	gallons

Inlet
72-inch reinforced-concrete pipe in terminal reach of South Bay Aqueduct feeds 11-foot-diameter inlet riser inside of terminal reservoir—top of riser, elevation 475 feet

Outlets
Domestic supply, 54-inch pipeline at base of tank with 54-inch butterfly shutoff valve—control at treatment plant by water user; ground water recharge, through 60-inch and 54-inch emergency overflow pipeline—control, 18-inch butterfly valve at base of 11-foot-diameter riser in tank—overflow when water surface in tank rises above elevation 475 feet is discharged to ground water percolation basin; 8-inch-diameter drain in bottom of reservoir empties into overflow pipe

Geology and Soils

Geology

South Bay Aqueduct crosses the northerly portion of the Diablo Range, a mountain range that consists mainly of sedimentary rocks folded into northwest-trending anticlines and synclines. Active faults are present in both the east and west parts of the mountains; these faults also trend northwest.

On the east side of this conveyance reach, Cretaceous sandstones and shales of the Panoche formation are folded into a large anticline, the Altamont anticline. This large fold forms the hills around the east margin of Livermore Valley. Younger, less-deformed, sedimentary rocks are encountered by the Aqueduct through Livermore Valley. West of Livermore Valley, folded Tertiary clayey gravels, sandstones, and siltstones in the Diablo Range are penetrated by Mission and La Costa Tunnels. Along the west flank of the Diablo Range, the remainder of the conveyance system passes through Tertiary sedimentary rocks and residual soils.

Soils

Soils vary in texture, chemical properties, and physical characteristics. Residual soils, formed in place by natural weathering, occur in the higher hills and usually are of shallow depth. Alluvial soils vary from clays to gravels, depending upon the nature of the deposition and the geology of the parent rocks Some soils, developed under conditions of poor drainage, contain accumulations of soluble salts which produce isolated pockets of high-sulfate ground water Such soil pockets were either overexcavated or re placed with suitable materials and/or the adjacen facilities were protected by anticorrosion measure including sulfate-resistant cement.

Seismicity

Seismic activity is considered variable along South Bay Aqueduct from the intake area near Tracy, where the seismic potential is low, to the terminal facilitie near San Jose, where seismicity is relatively high South Bay Aqueduct crosses two major active faults the Calaveras fault near Sunol and the Hayward faul near Warm Springs. The Aqueduct is aligned along o near the Hayward fault from Warm Springs to th terminal facilities. The Aqueduct also crosses the les extensive and probably active Greenville fault, jus south of Altamont (Figure 28).

The Calaveras fault branches from the San Andrea fault near Hollister and extends northward to the vi cinity of San Pablo Bay. Earthquakes have originate along the Calaveras fault, and displacement may hav occurred in 1861. The Hayward fault splits from th Calaveras fault about 15 miles southeast of San Jos and also extends northward to the vicinity of Sa Pablo Bay. Displacements occurred on the Haywar fault during the "great" earthquakes of 1836 and 186 Surface rupture extended from the vicinity of Sa Pablo in 1836 and from San Leandro in 1868 south t the vicinity of Warm Springs. The ruptures were n traced south of Warm Springs. Evidence that fau creep is occurring on the Hayward fault in sever locations from Berkeley to Irvington has been repor ed. Creep has been observed as far south as Irvingto about 5 miles north of Warm Springs.

Between 1932 and 1965, 21 earthquakes with Ric ter magnitudes greater than 4.0 occurred within miles of the South Bay Aqueduct alignment. The hi torical record includes four earthquakes which wou have caused damage in the area of the South B Aqueduct. These earthquakes originated in 1836 a 1868 on the Hayward fault, in 1906 on the San A

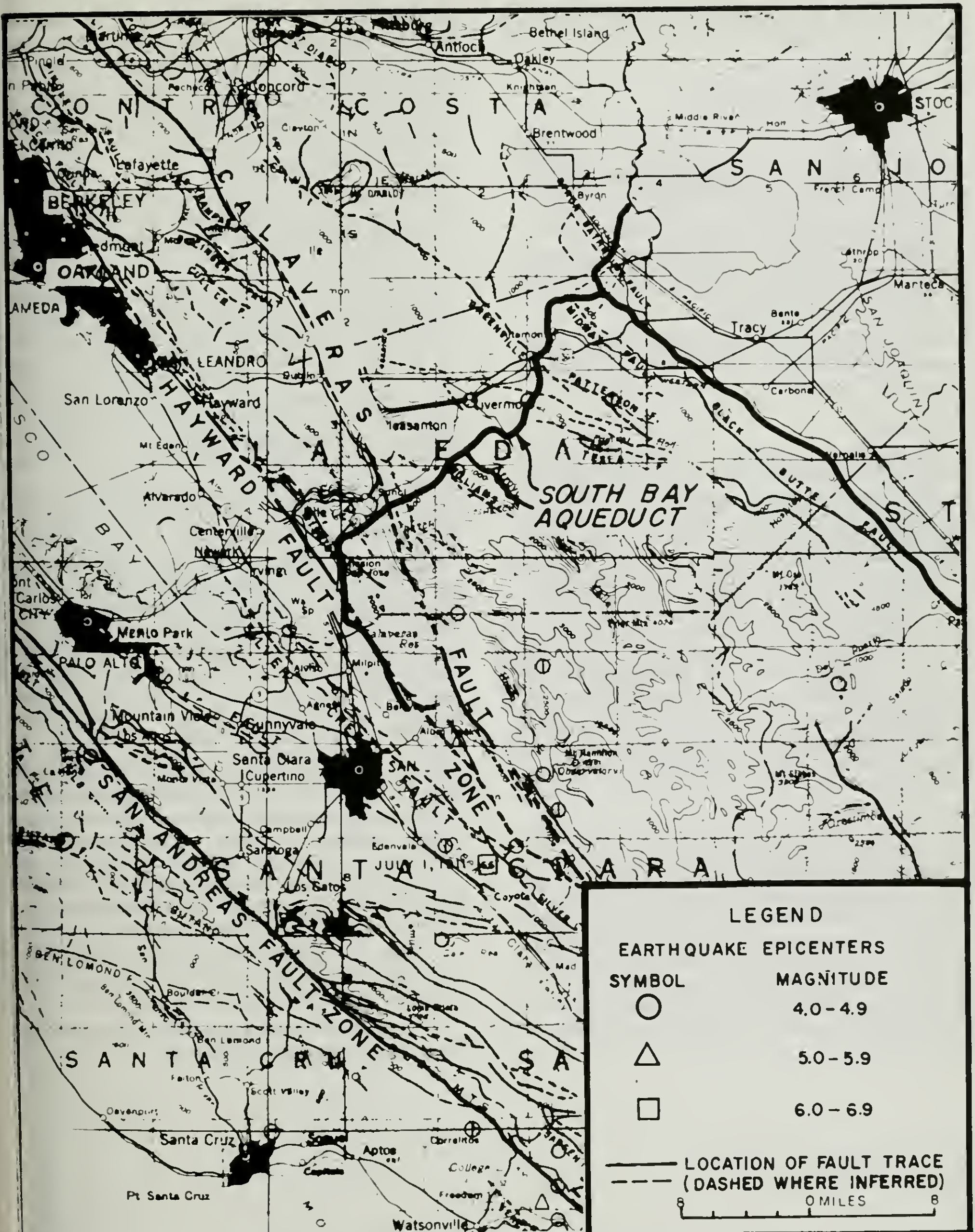

Figure 28. Earthquake Epicenters and Faults Near South Bay Aqueduct

dreas fault, and in 1911 near Evergreen from movement not specifically attributed to any one particular fault.

Design

Criteria

General concepts and criteria used for design of the conveyance facilities of the State Water Project are discussed in Chapter I of this volume.

South Bay Aqueduct largely was designed prior to the development of these criteria. In many instances, experiences gained during design and construction of the South Bay Aqueduct system were later applied to the California Aqueduct system.

Because of this relationship, design criteria are discussed more extensively in this chapter than in other chapters of this volume.

Economics

The alignment and sizing of the pumping, storage, and conveyance features of the South Bay Aqueduct system were developed using the "variable N" method. In this method, the "N" value, or value of an incremental foot of head at a pumping plant, is the present worth of all added capital and operating costs during the life of the plant(s), or other specified period of years, due to an addition of 1 foot of head.

This value was used to determine the most economical aqueduct design since, at the optimum design, any further saving in aqueduct and storage feature costs would be balanced by an increase in plant costs.

Plans and specifications were prepared using alternative bidding schedules for pipeline work in order to develop more competitive bidding and reduce costs.

Canals

The canal design provided a 15-foot road on the right side of the canal and a drainage berm on the left side (Figure 29). The height of the roadway above the top of the lining varied from its normal 1.5 feet in height to as much as 20 feet in deep cut sections. Experience has since indicated that, from a maintenance standpoint, it was worth the additional capital expense to provide a generally level, paved, primary, operating road slightly above the lining and also to provide a similar secondary road on the opposite side of the canal.

Lining. The canal sections were lined with unreinforced concrete from 2½ to 3½ inches thick normal to the surface. Canal design base width was 8 feet with side slopes of 1.5:1. Longitudinal grooves were provided near the intersection of the curved portion of the invert with the side walls. Transverse grooves were spaced at approximate 7-foot - 6-inch centers to control cracking (Figure 30).

The Livermore Valley Canal section was used to test several types of impermeable sublining. The details and findings of these tests are discussed later in this chapter.

Ground Water. In order to limit water pressure behind the canal lining from high ground water or operational drawdown, underdrains and flap valves were installed along selected portions of the canal reaches. Underdrains consist of a 4-inch layer of mineral aggregate beneath the lining. Flap-valve assemblies placed in a 2-foot-square pocket of gravel allowed water to flow only into the canal. The final location of these valves was determined during construction.

The use of these valves has since been discontinued because they are difficult to place and they interfere with canal cleaning. Open joints are used instead.

Siphon and Check Transitions. Open-channel transitions have a smooth and gradual taper to minimize head losses. A 30-degree angle of convergence of the side walls with the centerline of the structure for the inlets and a 22½-degree angle of divergence for the outlets were selected. To minimize head losses, the length of rectangular-to-round transitions for siphons and pipelines are slightly greater than twice the diameter of the round section (Figures 31 and 32).

To prevent blowback of entrained air in the inlet leg of siphons, the following criteria were adopted:

1. The combination of the inlet pipe slope, pipe diameter, range of "n" values, and flow rates that may be encountered was designed to fall on or below the curves of the critical Froude numbers as defined in Mr. Robert Sailer's May 1955 Civil Engineering Article entitled "San Diego River Aqueduct".

2. Siphon inlet legs were placed on a straight alignment and uniform grade from the inlet to a point at or beyond the downstream static pool level.

3. Siphon inlets were formed for smooth flow and avoidance of local air entrainment.

Siphons. Canal siphons were selected for crossing designated county roads and stream channels. Optimum size for the siphon barrels was determined by comparing the cost of the structures with the value of the attendant head loss. Precast pipe of the same size was used for uniformity and economy, wherever possible.

Check Structures. Two different check-structure configurations were adopted because of variable operational requirements at various locations.

Checks located immediately ahead of a single-barrel pipeline are single-gated structures to permit narrow transitions to the pipes. Checks at siphon entrances were equipped with side-channel spillways for operational flexibility during the period of increasing water demand. Spillway capacities are 120 cubic feet per second (cfs) with spillway stoplogs removed and check gates closed (Figure 32).

Checks located in open-canal sections were equipped with two radial gates and stoplog slots which allow servicing of one gate at a time (Figure 33).

Except for Dyer and Del Valle check structures, check structures were designed to pass the full canal capacity (300 cfs) over the closed gates within the canal-bank freeboard. Siphon spillways located upstream of Dyer and Del Valle check structures were designed to spill the water out of the canal before the canal bank would overtop (Figure 34).

Float wells are located adjacent to check structures to allow remote monitoring of water surface elevations (Figure 35).

Cross Drainage. Cross drainage is conveyed under the canal in culverts, over the canal in pipe or box overchutes, or into the canal (Figures 36 and 37). Choice of disposal method was determined by topographic conditions at the site. The capacity of the structures was designed based on information and data provided by the Alameda County Flood Control and Water Conservation District and/or charts contained in the California Division of Highways' (now the Department of Transportation) publication "California Culvert Practice" (Second Edition).

Road Crossings. Farm and county road crossings were provided by bridge overcrossings or siphon undercrossings.

Farm road bridges have a roadway width of 20 feet to accommodate farm machines. They were constructed of timber deck with concrete footings and were designed for H-15 loading.

County road crossings utilize reinforced-concrete bridges designed as single-span structures with cast-in-place pile-supported abutments. These bridges were designed for H-20 loading and have a T-beam-type deck varying in width from 28 feet to 40 feet with a girder spacing of 7 feet - 4 inches.

Siphons were selected where the county right of way was wide and the roadway later might be widened or where the road crossing could be combined conveniently with adjacent stream crossings.

Utility Crossings. All pipeline overcrossings are single span except one, where a center pier is used.

Vertical clearances for pipelines were established as follows:

1. Three-foot minimum cover for conduits beneath the canal.
2. One-foot minimum between bottom of pipe and top of lining for pipelines supported above the canal.
3. Two-foot minimum cover for conduits under the operating road and berms.

Overhead power and communication lines were relocated by the responsible utility agency prior to aqueduct construction. Vertical clearances complied with State Division of Industrial Safety and Public Utility Commission codes.

Safety. Safety racks were provided upstream of siphon inlets and check-siphon transitions (Figure 38). Canal escape ladders were provided on alternating sides of the canal at 500-foot maximum intervals and at the headworks of siphons and pipeline reaches.

The entire right of way was fenced with woven wire fabric and three strands of barbed wire to a height of 4½ feet. Seven-foot-high chain-link fencing was provided around siphon inlets and outlets and radial gate structures. Pipeline crossings were provided with safety-screen barriers on both sides of the canal.

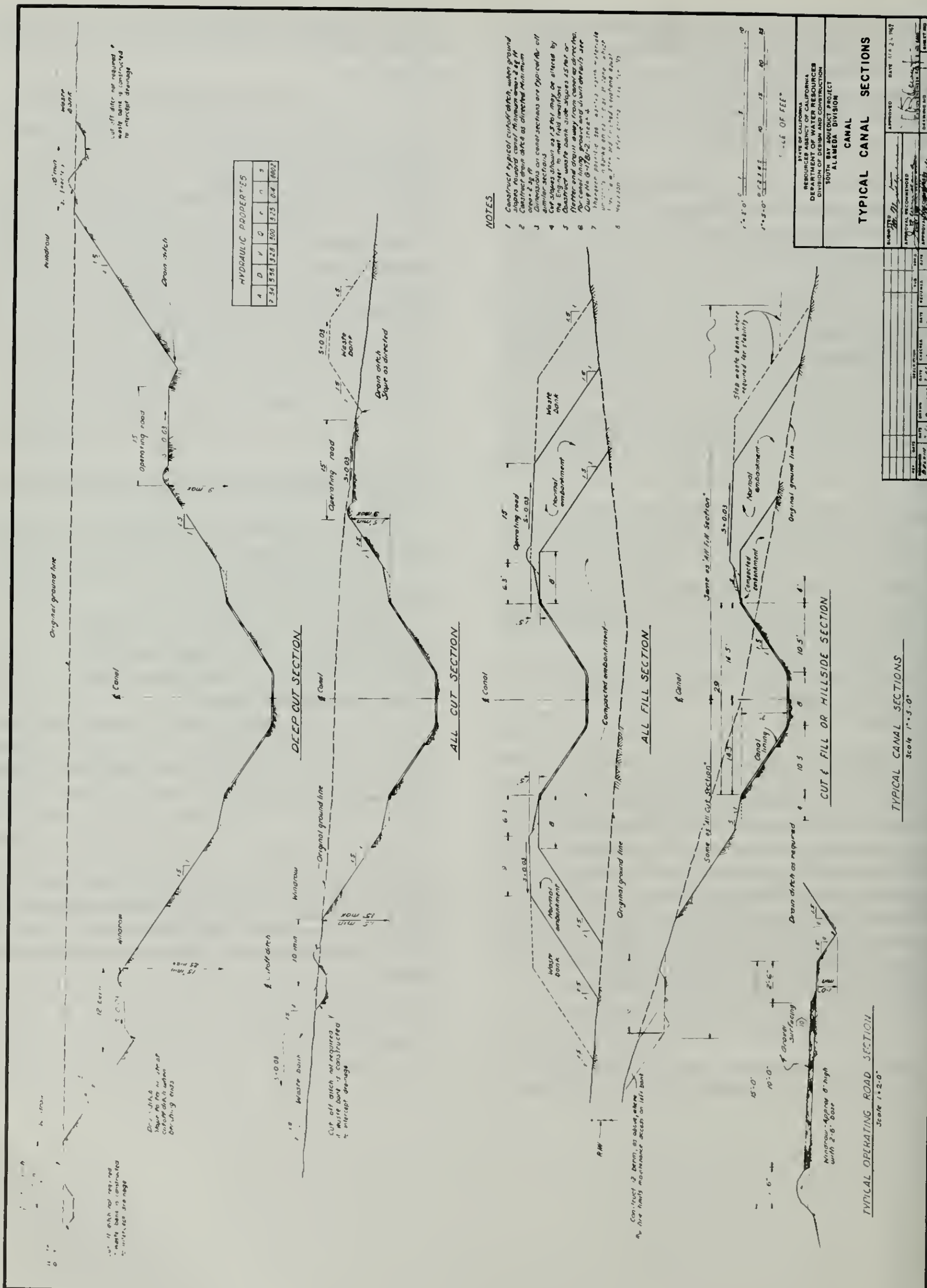

Figure 29. Typical Canal Cross Sections

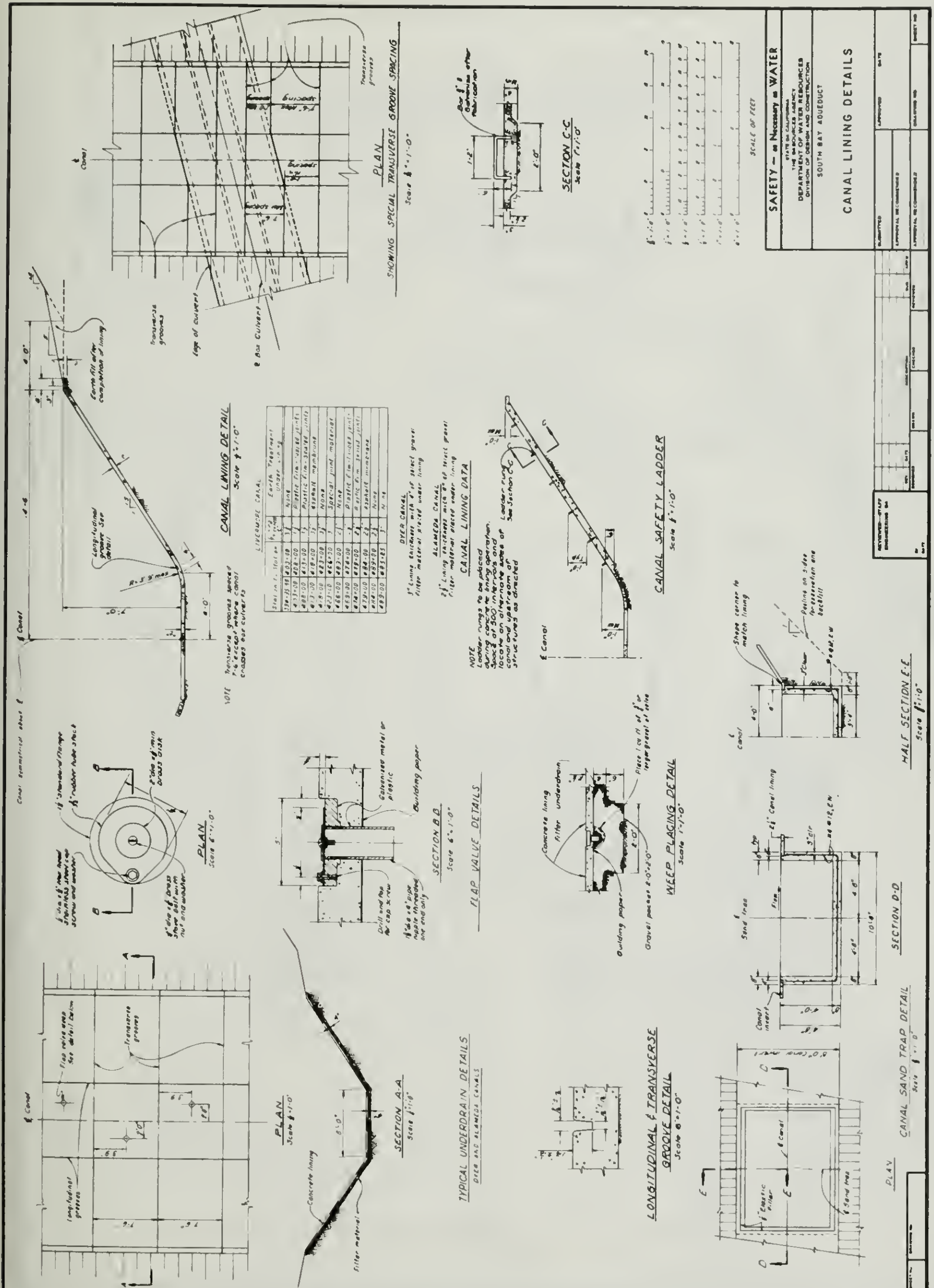

Figure 30. Concrete Canal Lining Details

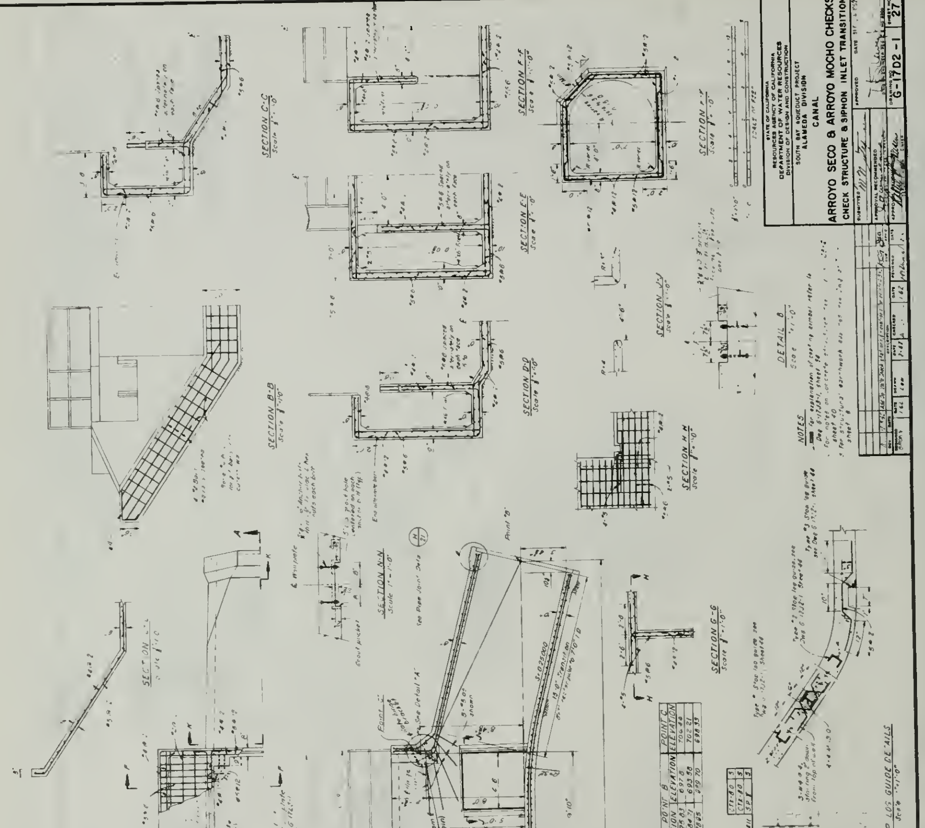

Figure 31. Check Siphon With Operational Spillway

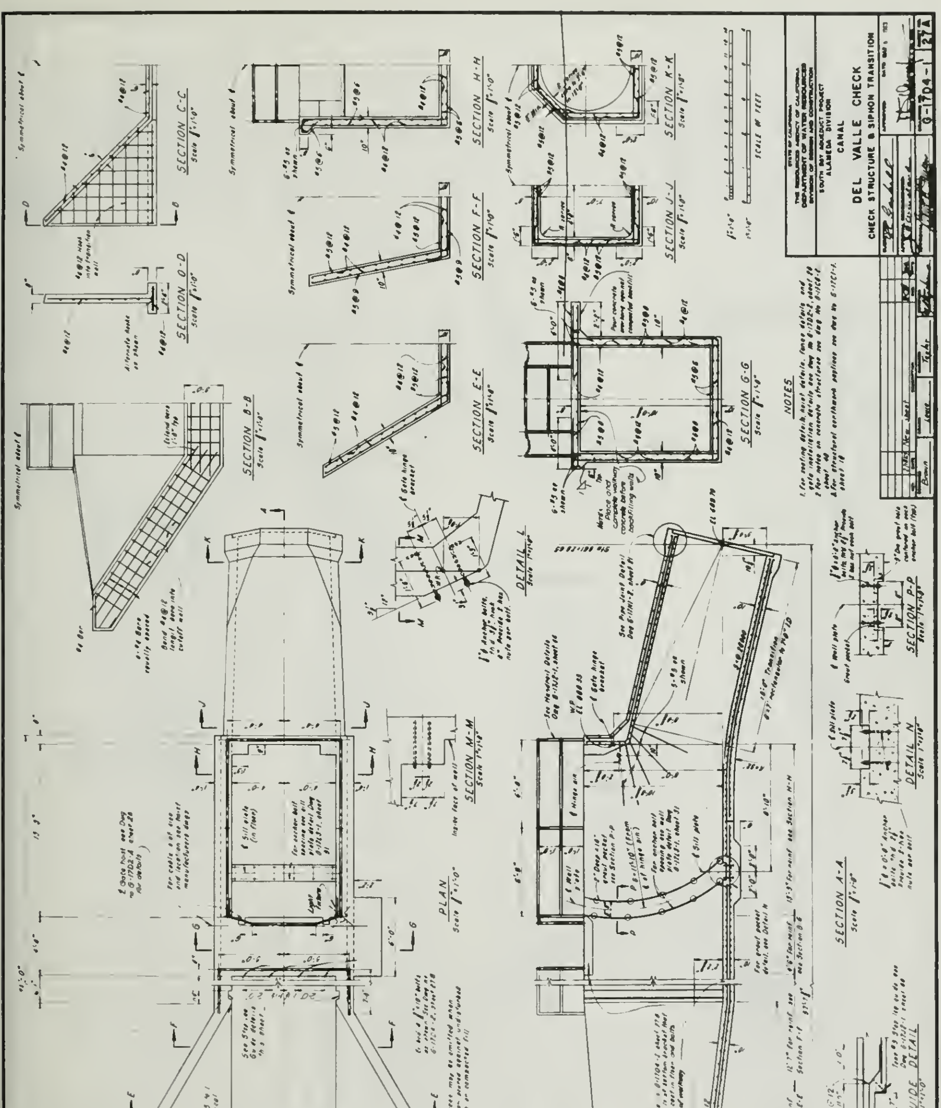

Figure 32. Check Structure and Siphon Transition

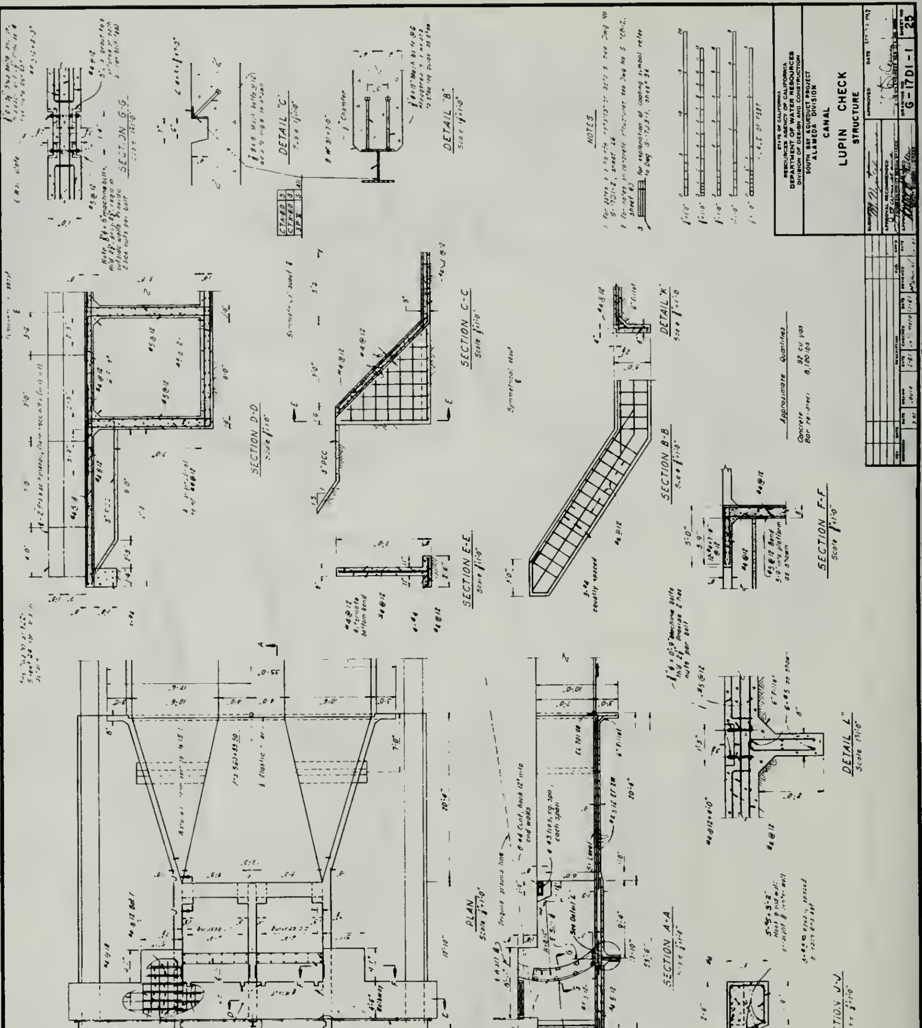

Figure 33. Two-Gate Check Structure With Overflow Spillway

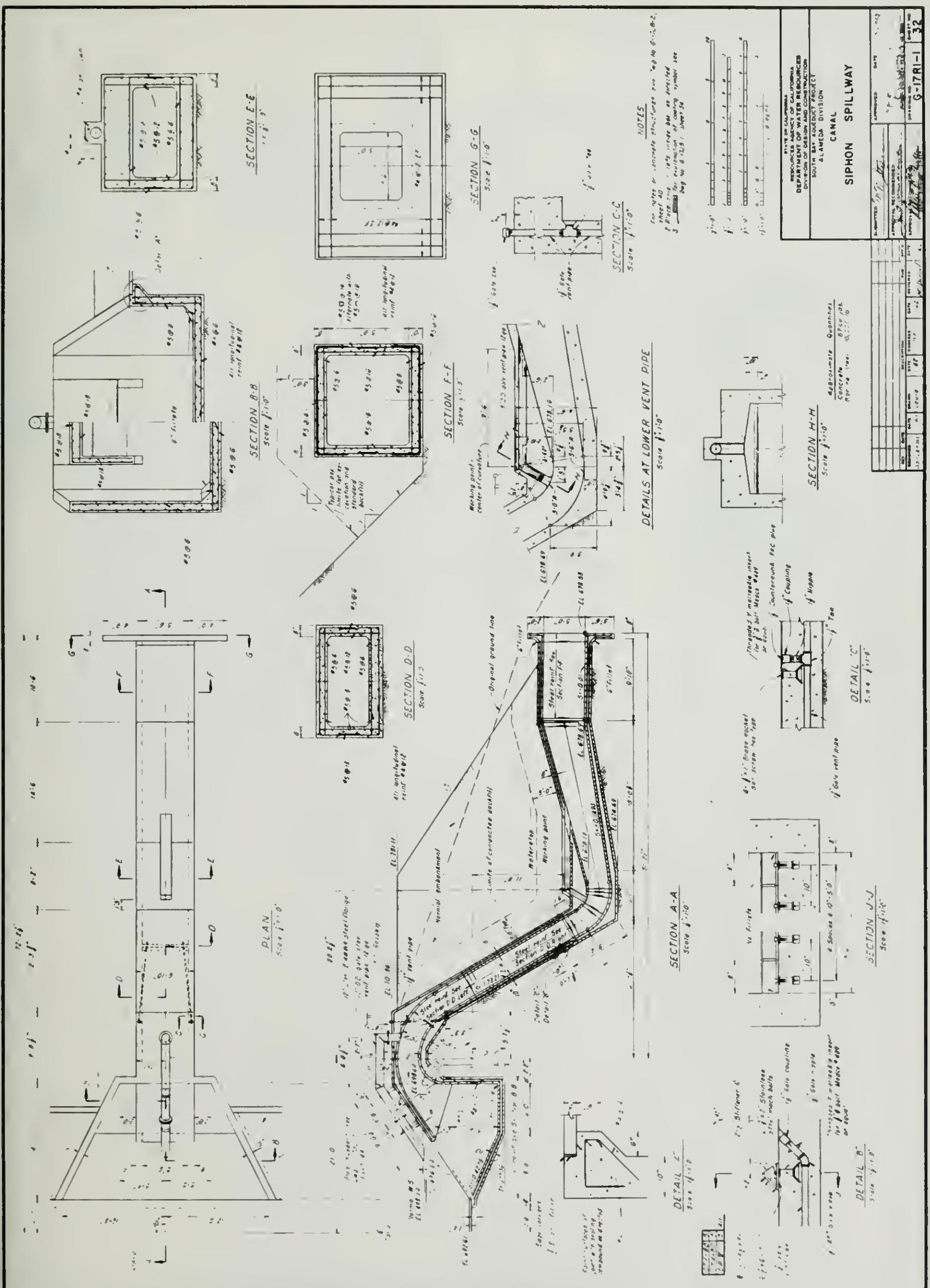

Figure 34. Siphon Spillway

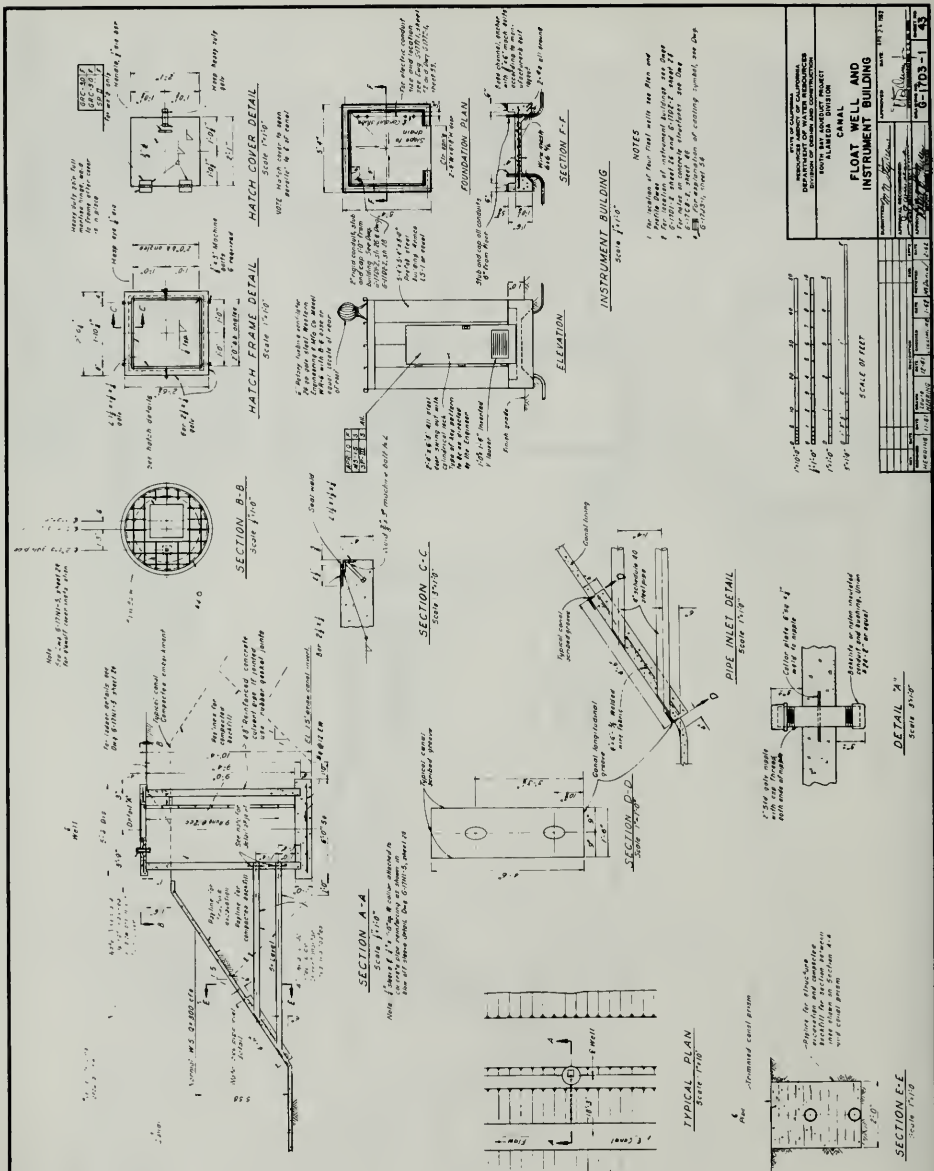

Figure 35. Float Well

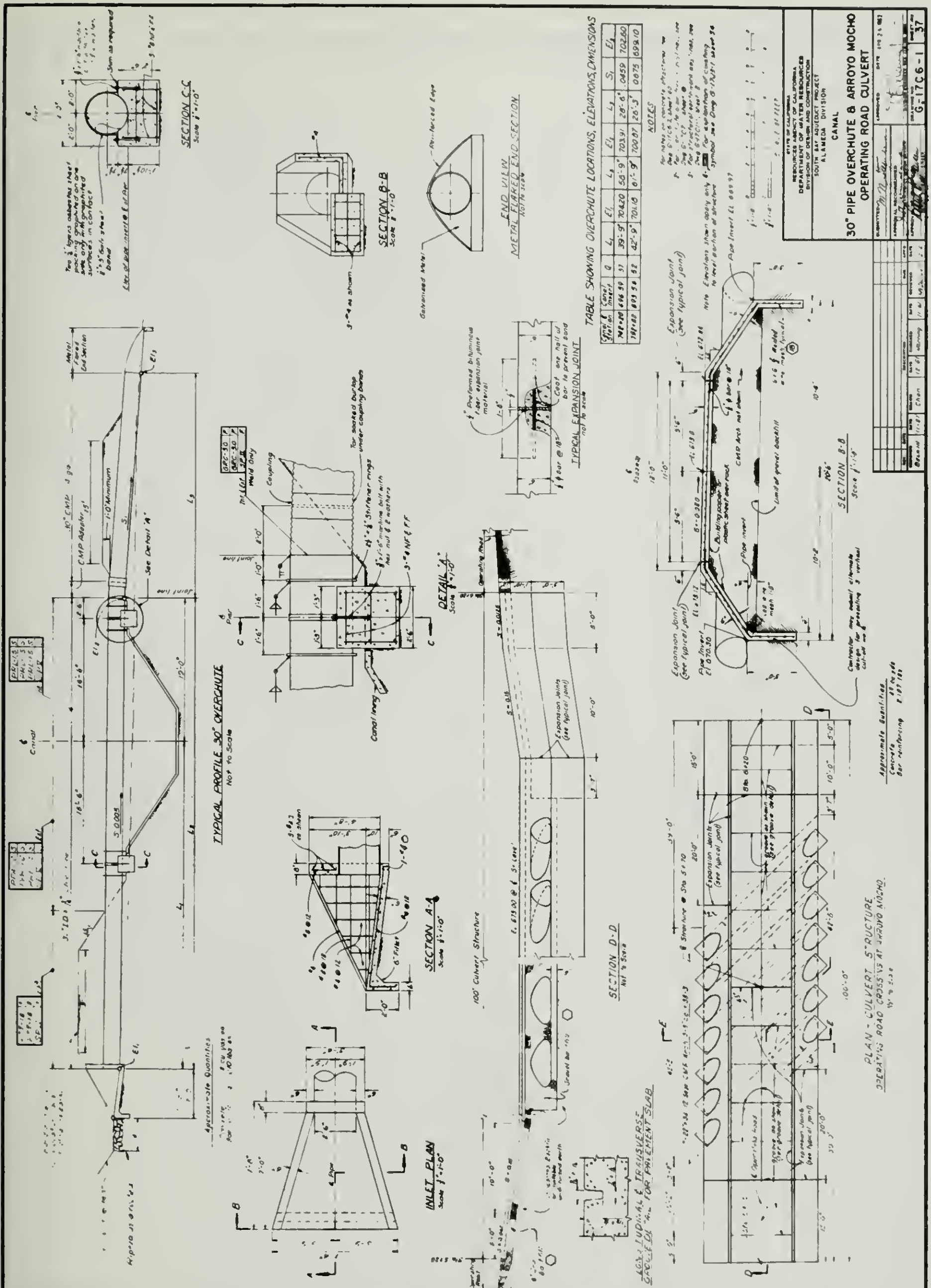

Figure 36. Pipe Overchute and Culvert Crossing

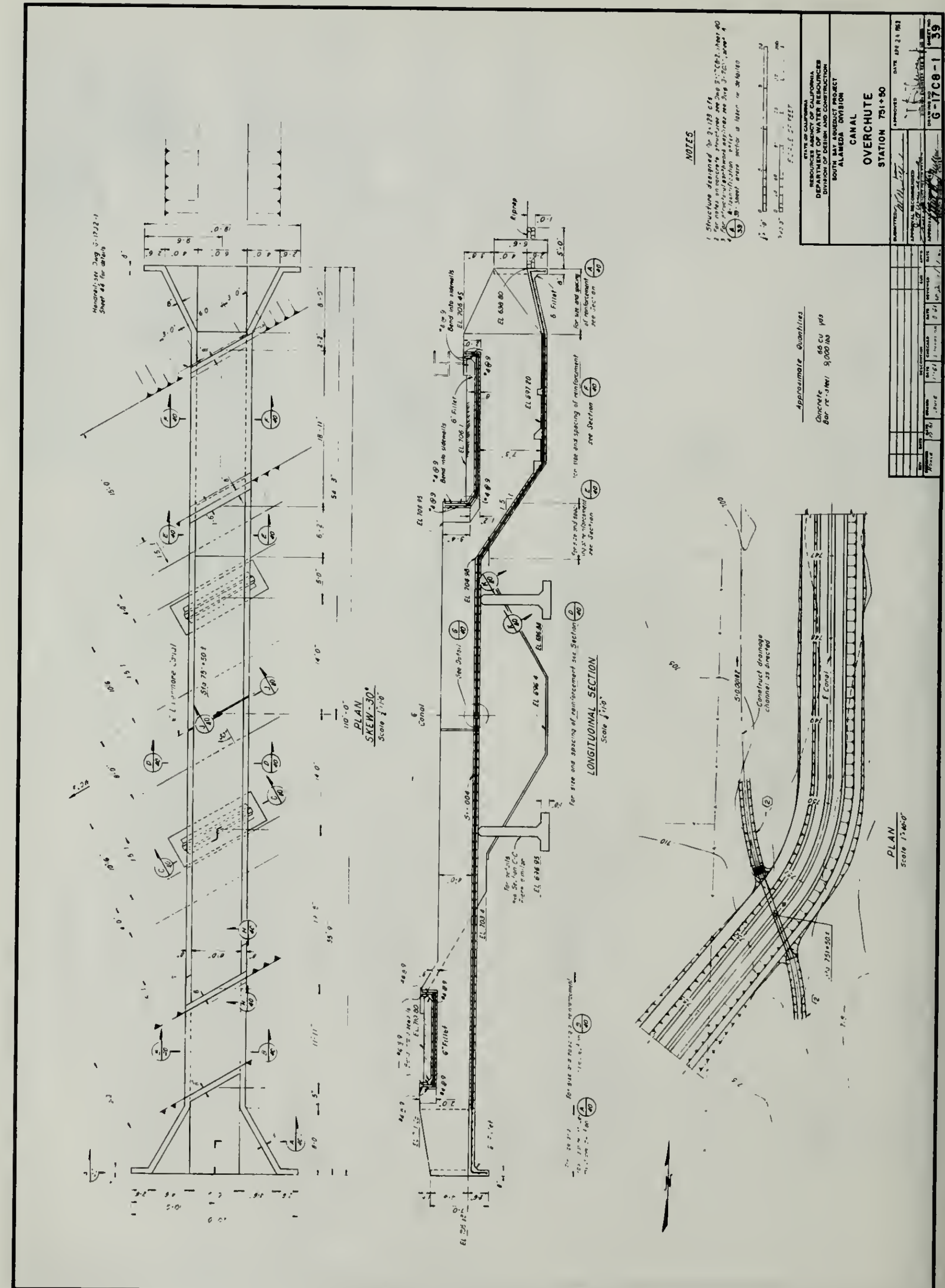

Figure 37. Box Overchute

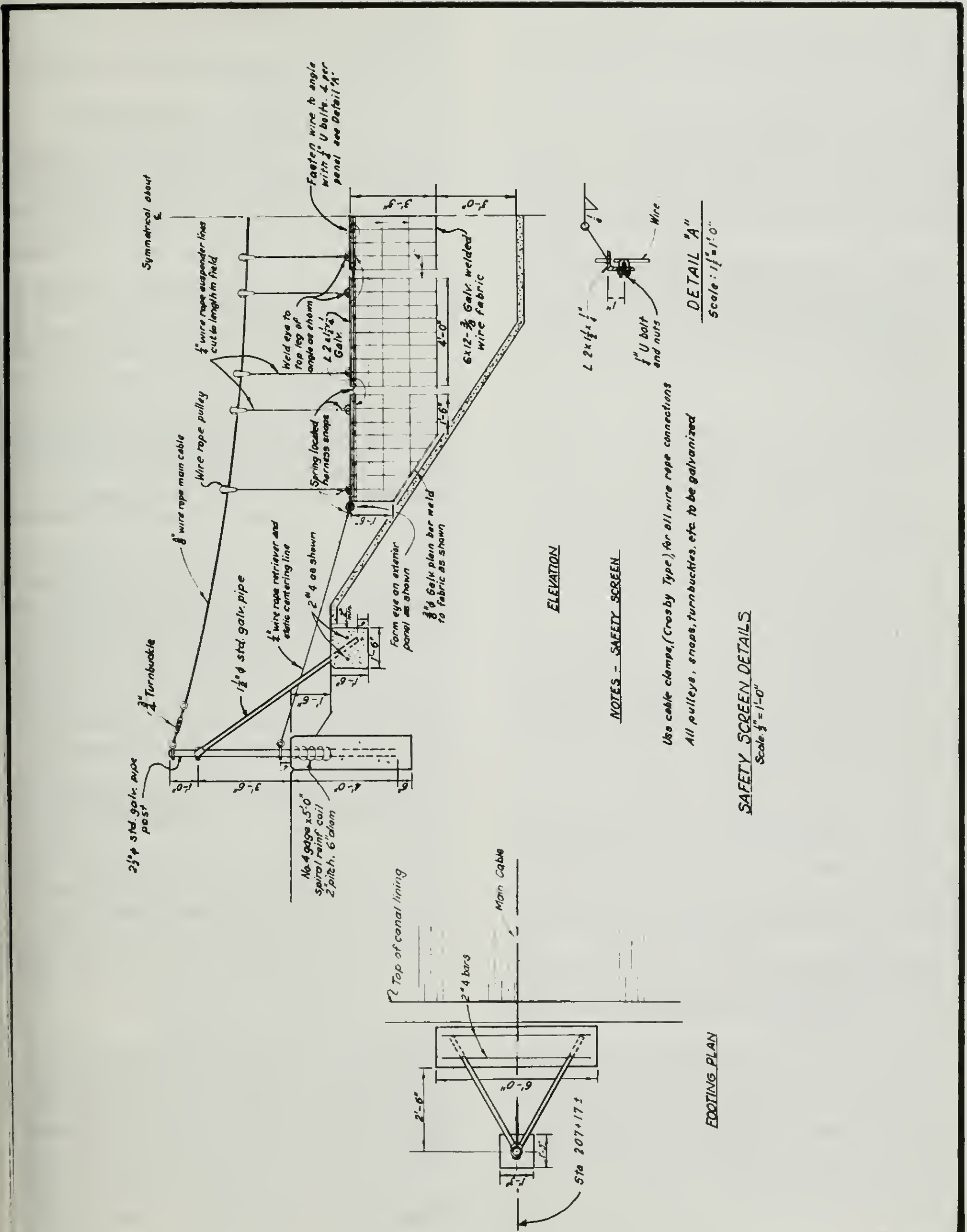

Figure 38. Canal Safety Racks

Pipe Selection

Pipeline reaches of South Bay Aqueduct were designed for (1) reinforced-concrete noncylinder pipe; (2) reinforced-concrete cylinder pipe; (3) prestressed-concrete, embedded, cylinder pipe; and (4) steel pipe. Alternative types of pipe specified for a specific reach depended on pipe size, external loading, and internal pressure. For heads greater than 125 feet, steel pipe or cylinder pipe was specified. Pipe diameters ranged from 42 inches to 90 inches. The following sections present the design considerations specified.

Reinforced-Concrete Pipe. General design criteria for all reinforced-concrete pipe were:

1. Minimum wall thickness was based on AWWA Standards C300-57 and 302-57.
2. Earth loads were determined by the following formulas:

$$W = D \times h \times A$$

$$A = B\ (100 + \frac{20h}{D}),\ \text{in which}$$

W = earth load on pipe, in pounds per lineal foot

D = outside diameter of pipe, in feet

h = height of earth above the pipe, in feet

A = maximum unit weight value for cross country = 150 pounds per cubic foot (pcf)
maximum unit weight value for road crossings = 180 pcf

B = 1.0 for cross country
1.2 for road crossings

3. External loading was based on earth covers of 7, 10, 15, and 20 feet. Since an earth cover of 7 feet over a 7-foot pipe was approximately equal to 3 feet of earth cover plus an H-20 highway loading, a 3-foot cover was the minimum specified.
4. Internal design pressure was for 50 feet of head (minimum) plus increments of 25 feet.
5. Moments and thrusts were determined using Olander's coefficient based on a 90-degree bedding angle. As a safety factor, a 120-degree bedding angle was specified.
6. Concrete design was based on a 28-day compressive strength of 4,500 pounds per square inch (psi).

Noncylinder Pipe. Design criteria for reinforced concrete noncylinder pipe were:

1. Reinforcing bars were intermediate grade with a 40,000-psi yield point.
2. The ultimate design method with a 1.8 load factor was used to combine hoop tension and bending.
3. The internal pressure force was computed using the inside diameter of pipe.
4. Steel stresses were computed by applying the direct load at the center of gravity of the two layers of steel, which initially was assumed to be equidistant between the two layers of steel.

Cylinder Pipe. Design criteria for reinforced-concrete cylinder pipe were:

1. Two-cage vertically cast pipe was specified with the inner cage including the steel cylinder.
2. Reinforcing bars were intermediate grade with a 40,000-psi yield point. Material for the steel cylinder was ASTM A245, Grade C, yield point 33,000 psi.
3. Ultimate design method with a 1.8 load factor was used to combine hoop tension and bending.
4. Allowable stress for hoop tension was 50% of the yield strength.
5. Internal pressure force was computed on the inside diameter of the cylinder.
6. Steel stresses were computed by applying the direct load at the center of gravity of the two layers of steel. The outer layer initially was assumed to be 33% of the total steel area.

Prestressed-Concrete, Embedded, Cylinder Pipe. Design of prestressed-concrete, embedded, cylinder pipe was based on the following:

1. Material for steel cylinders was ASTM A245, Grade C. Prestressing wire was ASTM A227, Class I or II.
2. Pipe was designed in accordance with the design equations and criteria contained in the Department's "Prestressed Concrete Cylinder Pipe Design Program 3044.55.2", which is based on American Pipe and Construction Company's Design Manual AP-D14, August 1962, and Robert Bald's article "Standardization of Design Procedures for Prestressed Concrete Pipe" from Journal of AWWA, November 1960. This design method generally corresponds to the 1964 issue of AWWA Standards C301 (Figures 39, 40, and 41).

Steel Pipe. Steel pipe was designed as flexible conduit, utilizing A283C steel having a minimum yield strength of 30,000 psi and an allowable design stress of 15,000 psi.

1. Earth loads were determined using Marston's formula for projecting conduits, $W = CwD^2$, in which

W = total earth load on pipe per lineal foot

C = load coefficient (projection ratio = 0 for consolidated or compacted backfill to top of pipe)

w = weight of earth per cubic foot, 100 pounds or 120 pounds for road crossings

D = outside diameter of conduit

Steel pipe was designed for a minimum cover of 7 feet and a maximum cover of 20 feet. Since an earth cover of 7 feet over a 7-foot-diameter pipe was approximately equal to 3 feet of earth cover plus an H-20 highway loading, 3 feet of cover was the minimum specified.

2. Minimum plate thicknesses for allowable steel pipe deflections were determined using Spangler's formula (ASCE Proceedings, Structural Division, 5T5, Volume 82, September 1956).

$$\text{Deflection} = \frac{fkwr^3}{EI + 0.061(er)r^3}\text{, in which}$$

r = radius of pipe, in inches

w = vertical load on pipe, in pounds per lineal inch

with f = 1.5 (deflection lag factor)

k = 0.1 (bedding factor)

(er) = 700

Backfill to the top of the pipe was specified as either granular material consolidated to 90% relative density or cohesive material compacted to 95% relative compaction.

Allowable deflections were limited to 2½% for pipe that was field mortar-lined after backfilling and 3% for shop coal-tar-lined pipe. When the earth cover exceeded 10 feet, vertical bracing was specified for mortar-lined pipe during backfilling.

3. Minimum plate thickness for internal pressure was based on Barlow's formula

$$t = \frac{pr}{se}\text{, where}$$

s = allowable stress was 15,000 psi

p = maximum internal design pressure (psi)

t = plate thickness, in inches

r = inside radius of pipe, in inches

e = joint efficiency for longitudinal joints

4. Minimum allowable plate thickness for handling

$$t\text{ (plate thickness, in inches)} = \frac{d\text{(normal pipe diameter)}}{300}$$

5. Minimum plate thickness to resist buckling and collapsing was established as 0.5% of the inside pipe diameter.

6. Reinforcement for air valve, blowoff, and manhole nozzles were designed according to ASME pressure-vessel codes. Design of reinforcement for larger openings was based on "Design of Wye Branches for Steel Pipe," Journal of AWWA, June 1955 (Figure 42).

Pipelines

Trenching and Backfilling. On the longer pipeline contracts, excavation for pipe trenches for bidding purposes was classified by difficulty of excavation and soil type. Related specifications for backfilling the trenches were developed. In rock or wet ground, 9 inches of overexcavation and replacement with compacted backfill were required. Compacted backfill was required where the pipeline was located on slopes steeper than 3 vertical to 10 horizontal. Consolidated backfill was required where the pipeline was located on slopes 3 vertical to 10 horizontal or flatter. At road crossings, compacted backfill was required to the level of the road base or as directed (Figure 43).

Access Structures and Manholes. Access structures were provided at blowoff and air valve locations for maintenance and operation. A typical structure access consists of a reinforced-concrete culvert pipe installed vertically over the pipeline and equipped with hinged metal covers. The inside diameter varies from 48 inches to 60 inches.

Manholes were provided at a maximum spacing of approximately 2,000 feet. Where possible, they were combined in access structures with air valves and blowoffs; otherwise, the manholes were buried and their locations marked with 6-inch by 6-inch redwood stakes. The manholes were designed using AWWA standards, except that 20-inch-diameter pipe was specified. This minimum size was increased in later designs to 22 inches. Many of the blind-flange manhole covers were equipped with corporation cocks to monitor changes in the hydraulic gradeline.

Blowoffs. Blowoff structures were provided at low points in the pipeline for dewatering. Each blowoff was placed in an access structure and includes a vertical riser pipe, a cast-iron tee with a top blind flange, a valve, and a discharge pipe. The discharge pipe extends from the valve outlet flange, through the wall of the access structure, to a natural drainage way. Due to the failure of two tees on Santa Clara Pipeline during initial pipeline operation, a sleeve coupling was incorporated into the pipe where it leaves the access structure in order to provide articulation.

The blind flange on the tee allows total dewatering by pumping through a suction pipe inserted in the riser.

On the higher head pipelines along South Bay Aqueduct, lubricated plug blowoff valves were provided.

Energy dissipators were provided at blowoffs where the surrounding terrain does not accommodate natural energy dissipation or where the operating head is greater than 75 feet. Concrete anchor blocks on the extension pipes without energy dissipators prevent displacement from vibration and thrust (Figure 44). Experience has indicated that this blowoff dissipation is very efficient and economical.

Air Valves. Air valve structures with automatic combination air release-vacuum valves were provided at high points in the pipeline to admit air during subatmospheric pipeline pressures, to release air during filling, and to discharge air accumulations at summits. Whenever these high points were close to minimum operating hydraulic gradeline, pipe vents were used instead of air valves. Each combination air release-vacuum valve was sized to supply air quantities matching the water discharge when all blowoffs are operating simultaneously. Maximum design velocity through the air valves is 200 feet per second. Two-inch air release valves were specified. In most cases, combination air release-vacuum valves were provided.

In addition to the above requirements, 4-inch combination air release-vacuum valves were provided at sharp breaks in grade on the steel-pipe alternatives (Figure 45). Nonrising stem valves located below the air valves permit inspection and maintenance.

Vents and Standpipes. Vents equal to the diameter of the main pipeline were provided at all high points where the design hydraulic gradeline approached the pipe profile. Vents are reinforced-concrete pipe, except for a 32-foot steel-pipe vent in the Santa Clara Pipeline. Standpipes with a standard manhole are connected to the pipeline for air relief along flat sections of pipeline operating partly full.

Aqueduct Meters. Seven telemetered main-line flowmeters monitor flow along South Bay Aqueduct and are located as follows: in South Bay Pumping Plant discharge lines (one for each line), in Del Valle Pipeline at Arroyo Road, in Del Valle Pumping Plant discharge line, in 30-inch turnout pipeline from Del Valle Branch, in Santa Clara Pipeline at Scott Road, and at the terminal facilities overflow pipeline.

Each flowmeter consists of a flow tube, operating on the Venturi flowmeter principle, connected to a differential pressure-type recorder and transmitter.

These flow tubes and recorders are housed in concrete vaults for access and maintenance, except the Del Valle Branch Pipeline flow tubes which can be maintained from the ground surface. The recording devices for these latter flow tubes are located in separate aboveground installations.

Turnouts. Pipeline turnouts were located as requested by water users. Most of the turnout pipeline connections consist of reinforced nozzle or wye branch connections equipped with a blind flange or bumped head for later hookup. Several turnouts are combined with blowoff structures.

The Vallecitos turnout connection is uniquely located at a summit in the pipeline. The turnout structure consists of a pipe outlet extending from the bottom of a special vent structure.

All turnouts have a shutoff valve; a regulating valve; a Department of Water Resources-approved flowmeter, recorder, and transmitter; and a means of on-site calibration of the meter.

Emergency Flow Control Structures

Several emergency use features were designed for the South Bay Aqueduct system which allow spilling or diversion of in-transit flow should a downstream emergency condition, such as an aqueduct failure, or an unscheduled rejection at a large-capacity turnout arise. These features include: 300-cfs-capacity, self-priming, siphon spillways on both Dyer and Alameda Canals; a spillway at Vallecitos turnout; the 200-cfs-capacity, remotely controlled, Sunol blowoff; and the overflow bypass structure at the Santa Clara terminal facilities.

In addition to the above features, diversion of in-transit flow along the Aqueduct also can be accomplished by remotely controlling the operation of the Del Valle Branch Pipeline system and the 20-inch cone valve at Altamont turnout (Figure 27).

Design Changes

Economic factors, foundation conditions, and increasing water demands created a need for some notable changes during the design and early construction period. These changes are discussed briefly.

Pump Lift and Conveyance Facility. The original design of the Aqueduct contemplated a two-stage pump lift from Bethany Reservoir up the eastern slope of the Diablo Range. Water then would be conveyed by a 1.5-mile-long tunnel to a canal reach extending across Livermore Valley to La Costa Tunnel. Adverse geologic conditions, unfavorable ground water conditions, and comparative economics of a single- versus a two-stage pump lift caused this concept to be rejected.

Staged Construction of Brushy Creek Pipelines. Estimated water demands through 1990 and delay of Santa Clara County to contract for water delivery resulted in the decision to provide staged construction of the single-lift pumping plant and Brushy Creek Pipeline. However, unanticipated increases in water demand and the firming up of Santa Clara County's scheduled deliveries required construction of the second-stage facilities in 1965.

Terminal Facilities. Original plans contemplated a dam and reservoir as the terminal facility for the South Bay Aqueduct system. The reservoir (Airpoint) capacity would have been about 5,000 acre-feet and would have been located on Arroyo de Los Coches, north of the City of San Jose. Even though geologic conditions were known to be marginal, it was believed that, based on the information available, a safe economic dam and reservoir could be constructed. However, further investigation of foundation conditions confirmed unfavorable geologic and seismic conditions, and the plan was abandoned. Since no other suitable reservoir site was available, Santa Clara Pipeline was extended southward to Santa Clara County's eastside treatment plant site where a 7.8-acre-foot steel-tank reservoir was provided.

Doolan Branch Pipeline and Reservoir. This reach of the system was planned to serve southern Contra Costa County. When the County decided not to contract for water service, these facilities were not constructed. A stub was provided in Altamont Pipeline at the Highway 50 crossing should the branch pipeline be required in the future.

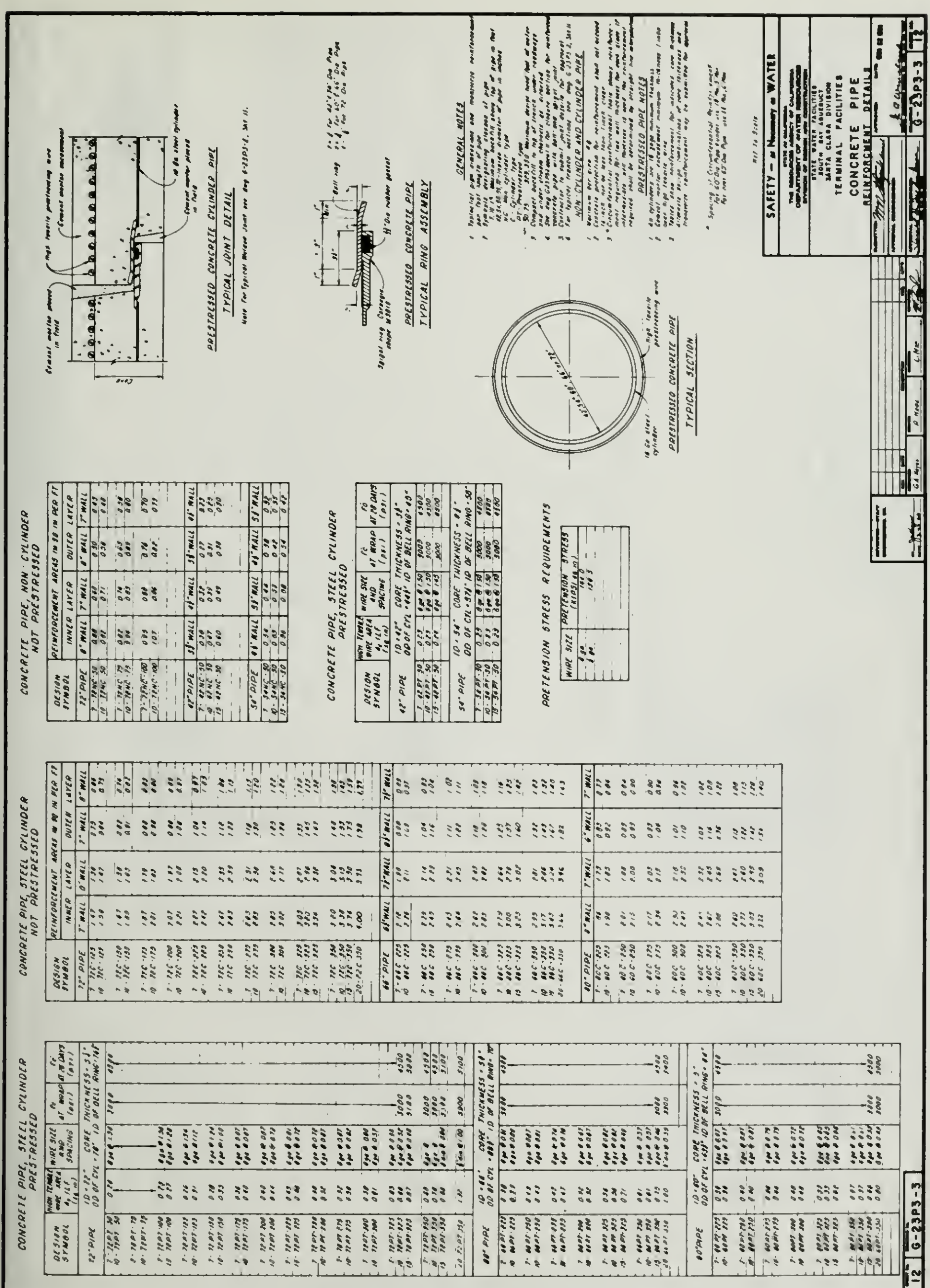

Figure 39. Concrete Pipe Reinforcement Details

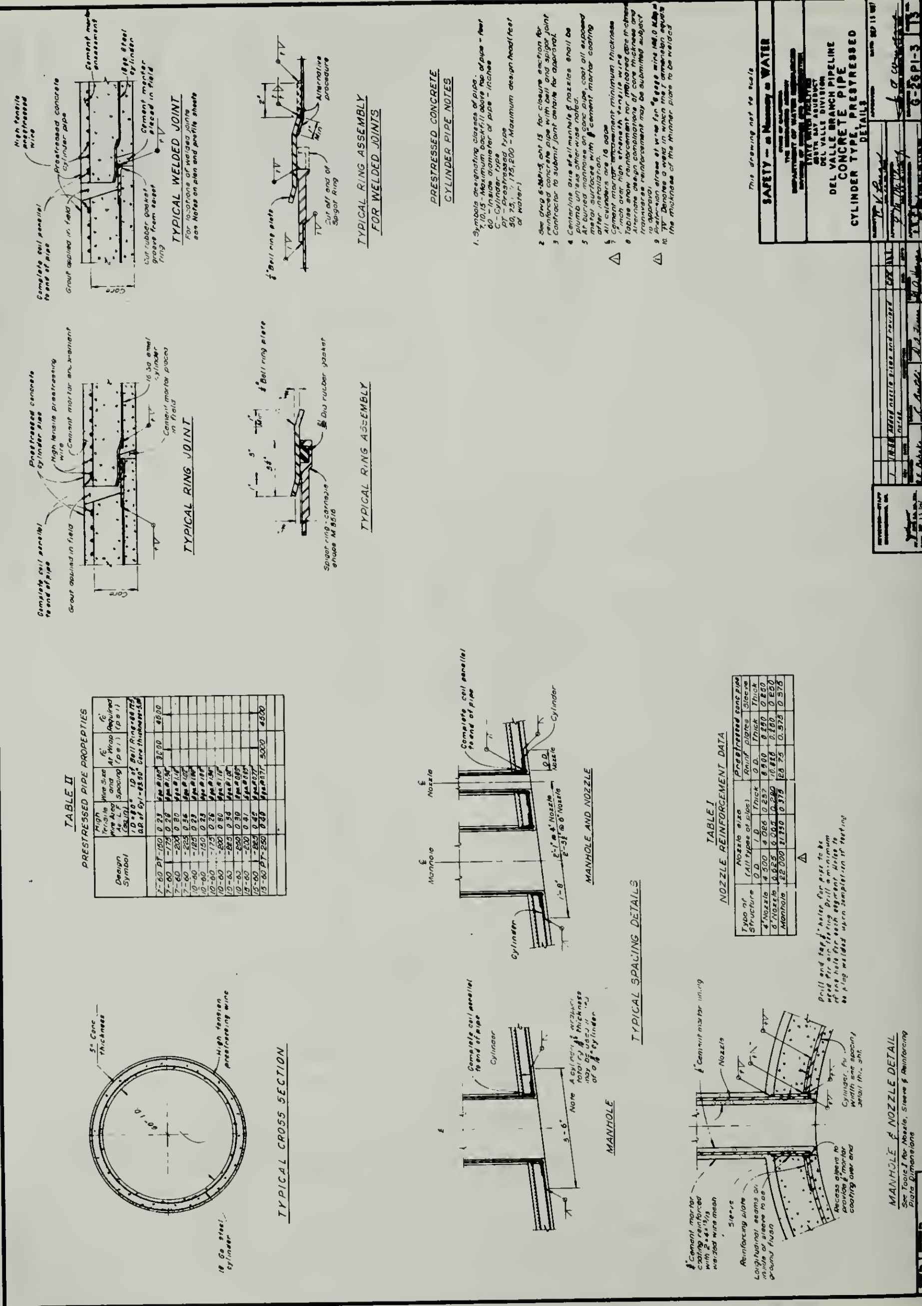

Figure 40. Prestressed-Concrete Pipe Nozzle Details

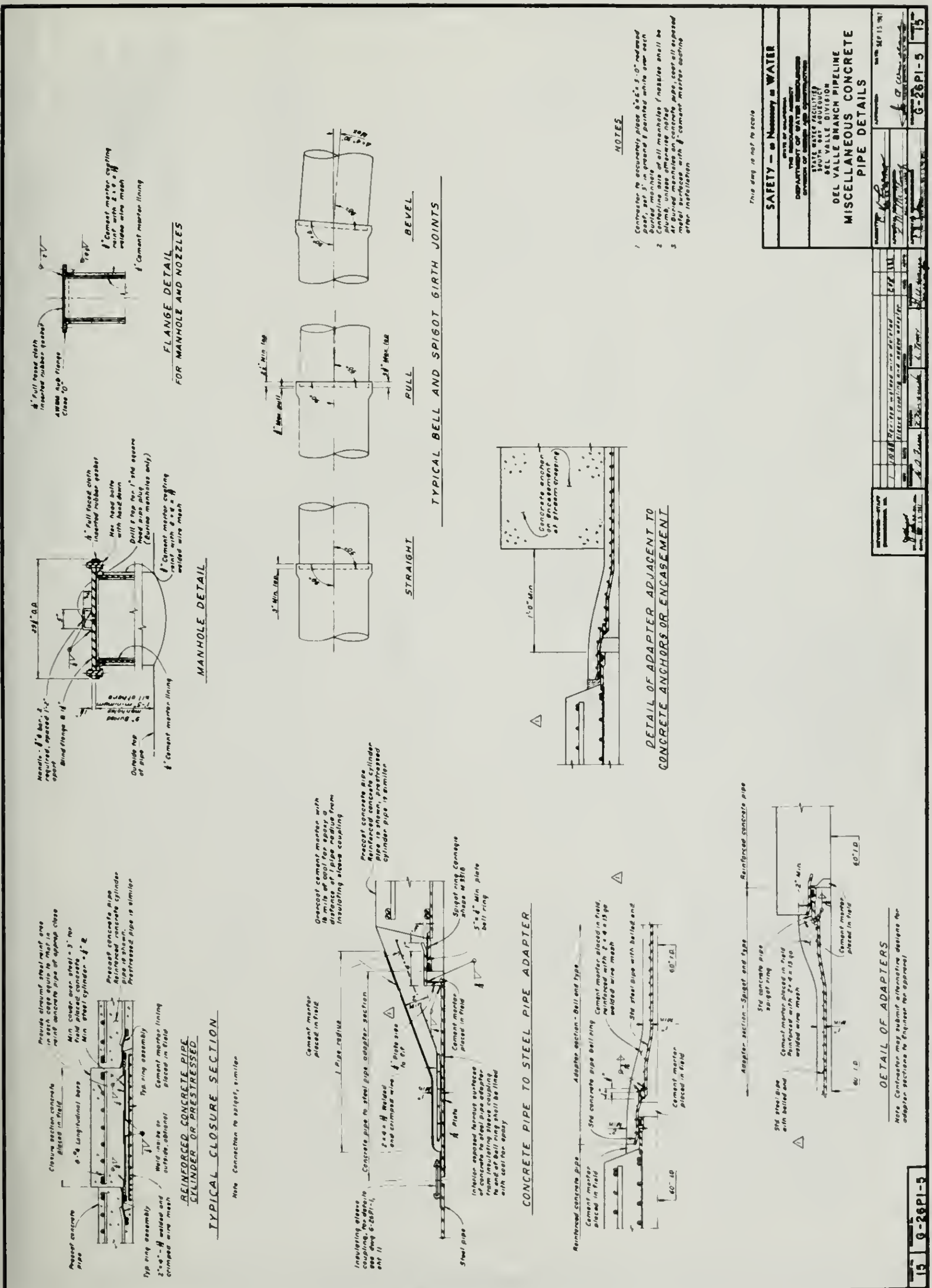

Figure 41. Concrete to Steel-Pipe Adapter Details

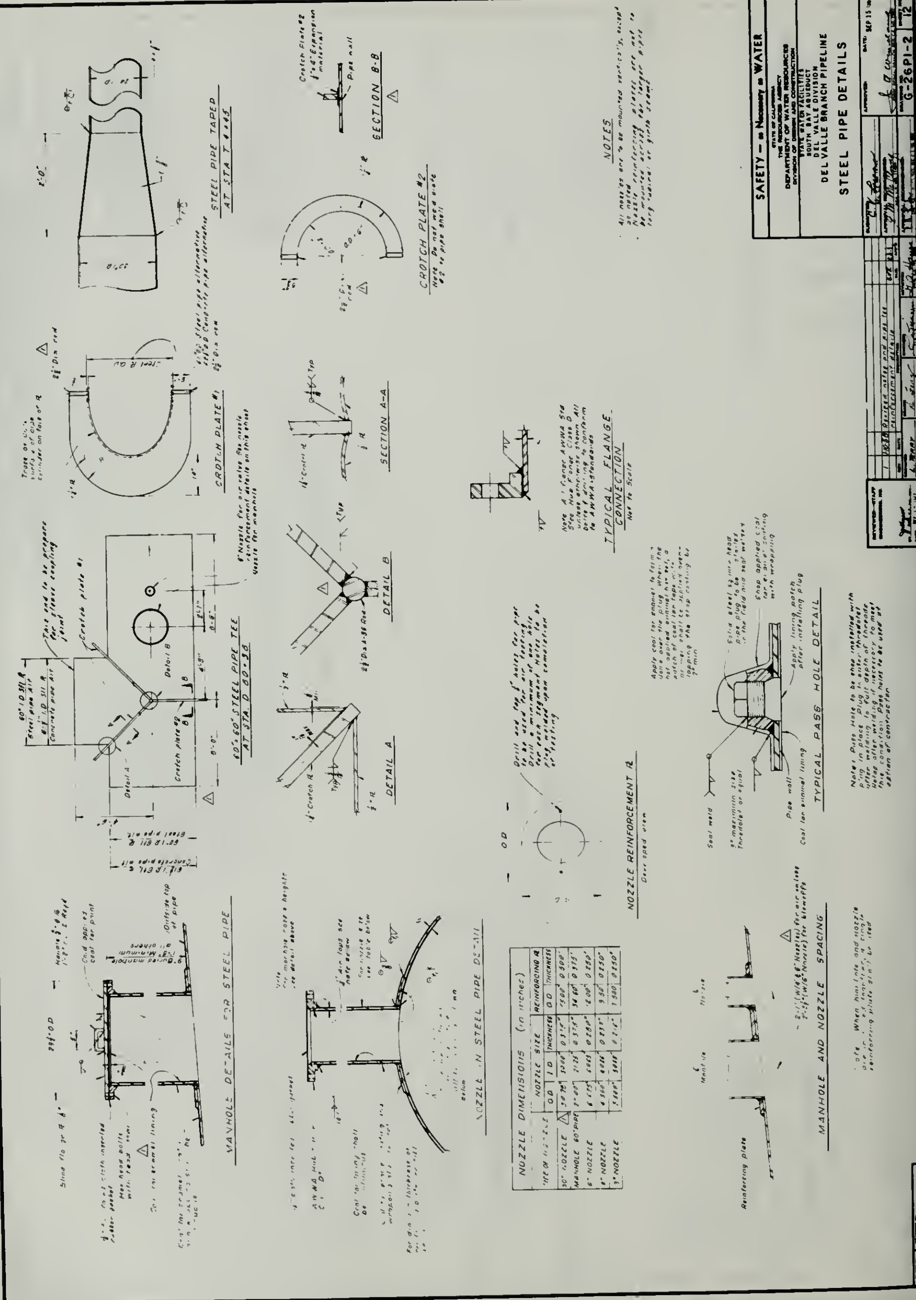

Figure 42. Steel-Pipe Branch Connections

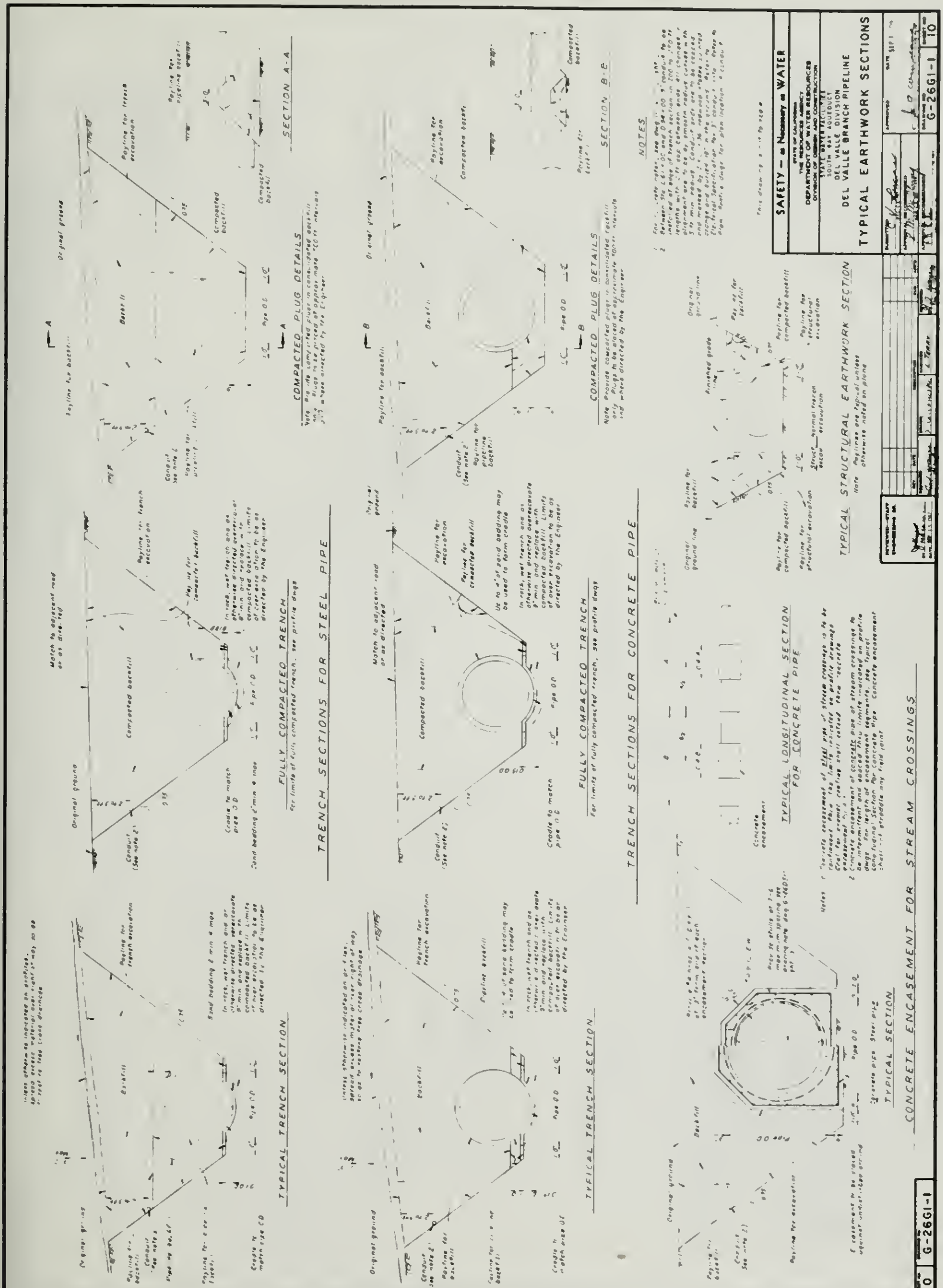

Figure 43. Pipeline Earthwork Sections

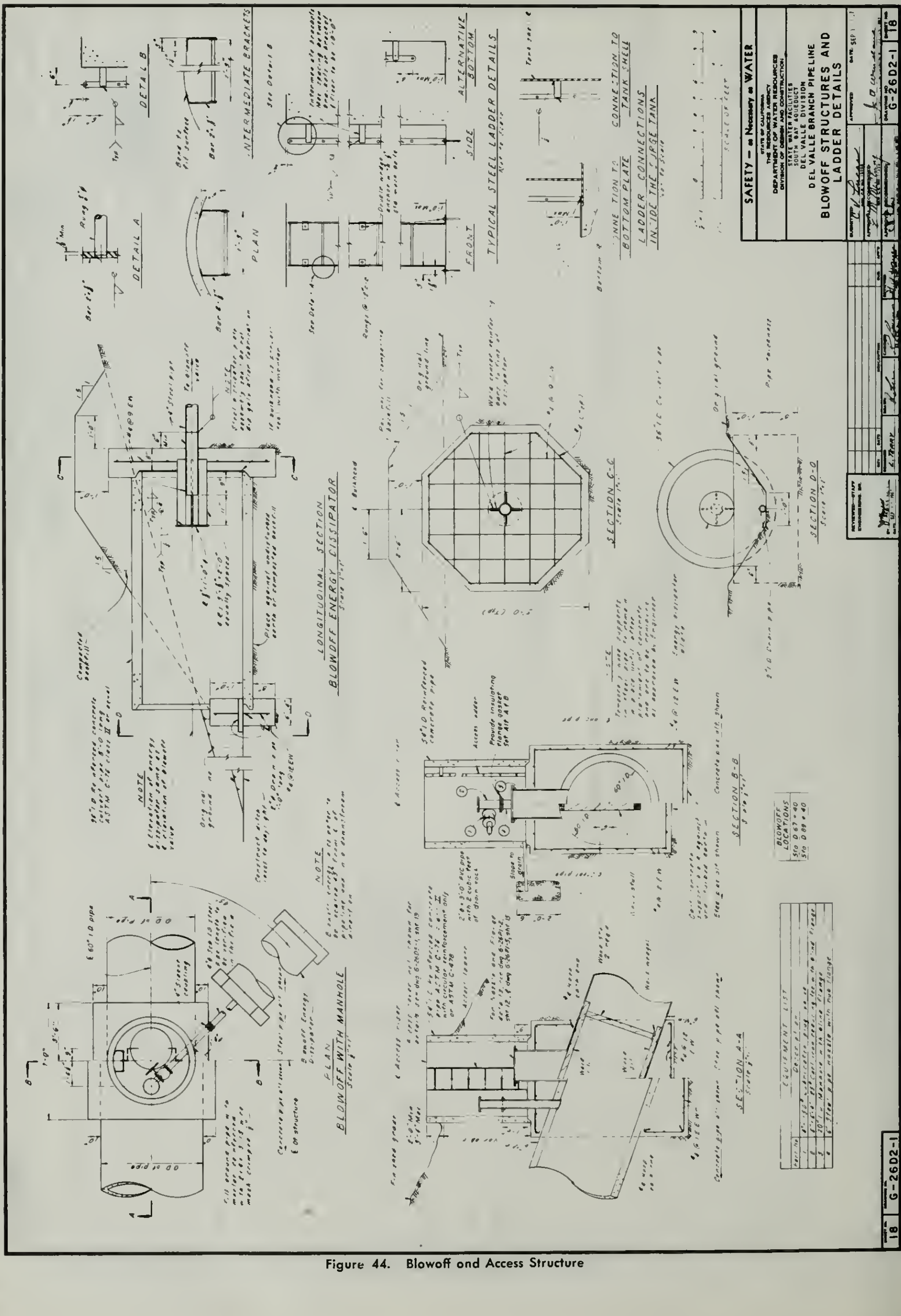

Figure 44. Blowoff and Access Structure

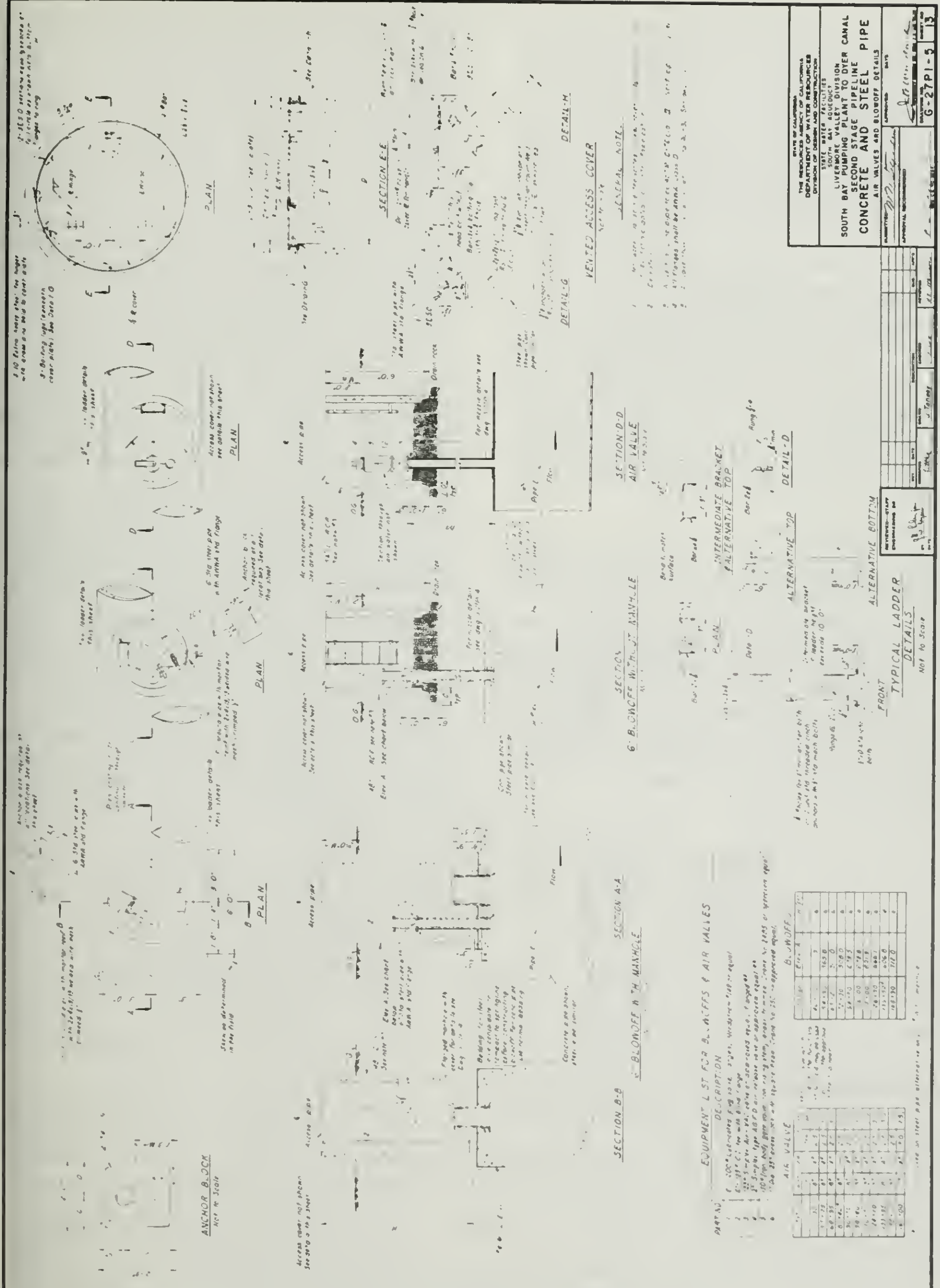

Figure 45. Air Valve and Blowoffs for Brushy Creek Pipeline

Construction

General information about the major contracts for construction of South Bay Aqueduct conveyance facilities is shown in Table 6. These facilities were constructed during the period from 1960 to 1969.

The project office for construction was located in Livermore and included materials-testing laboratory facilities. Field offices and laboratories were established to meet the geographical and technical requirements of the various construction sites.

Design and Construction by Contract Reaches

Surge Tank to Del Valle Pipeline

This reach extends from the surge tank for the initial stage of South Bay Pumping Plant through Alameda Canal. The second stage of Brushy Creek Pipeline was constructed under a separate contract. Supplemental contracts within this reach included the Dyer Canal check structure, a tunnel extension for Altamont Pipeline under Highway 50 (now Interstate 880), and repair and modification of Patterson Reservoir.

Brushy Creek Pipelines.

Design. These two lines convey water from South Bay Pumping Plant surge tanks to the back-surge pool at the entrance to Dyer Canal.

The first pipeline was designed as a 54-inch-inside-diameter, reinforced-concrete, cylinder pipe. Eleven 6-inch blowoff lines were provided at the low points. Ten 4-inch combination air release-vacuum valves were located at the summits of the Pipeline. The capacity of the line was established at 120 cfs with a velocity of 7.55 feet per second (fps). The hydraulic gradient is 0.00317 between the surge tank and the back-surge pool.

The second line was designed for a 225-cfs capacity with a velocity of 9.47 fps and has a hydraulic gradient of 0.00383. Ten 6-inch blowoff lines and nine 4-inch air valves were provided (Figure 45). This line was designed for three bidding alternatives: (1) 66-inch, prestressed-concrete, cylinder pipe; (2) 66-inch, reinforced-concrete, cylinder pipe; and (3) 67-inch, coal-tar-lined and coated, steel pipe. Of six bids received, four including the low bid were for alternative (1).

A back-surge pool, located at the head of Dyer Canal, was designed in conjunction with the surge tanks to provide a sufficient volume of water to prevent water column separations in the Pipeline, in the event of power failure at South Bay Pumping Plant. The back-surge pool water will maintain the Pipeline full during normal operation and during all stages of the surge cycle. An overflow weir was incorporated into the pool outlet to prevent Dyer Canal waters from flowing back into the Pipeline. Gravel side drains with collector pipes underneath the lining prevent uplift when the pool is suddenly drained (Figure 46).

Construction. Beginning at the outlet of the surge tank, this buried, 54-inch, reinforced-concrete pipe traverses 2.44 miles of rolling hills to the back-surge pool at the head of Dyer Canal. The rolling hills and steep slopes caused difficult construction problems.

The pipeline alignment was pioneered by benching out work areas with dozers. The trench was excavated with a backhoe equipped with a 2-cubic-yard bucket wherever possible. Hard rock was encountered along most of the length and was drilled and blasted.

A dragline was used to muck out the trench. Overbreak was excessive due to drilling below invert elevation and overloading the holes. Overbreak was replaced with compacted or consolidated material.

TABLE 6. Major Contracts—South Bay Aqueduct

	Specification	Low bid amount	Final contract cost	Total cost—change orders	Starting date	Completion date	Prime contractor
Surge Tank to Patterson Reservoir	60-15	$2,982,965	$3,584,164	$148,100	11/15/60	8/ 8/62	Case-Hood
Alameda Division Canal	62-12	1,317,787	1,420,018	73,747	8/ 7/62	8/27/63	McGuire and Hester
La Costa and Mission Tunnels	63-03	2,679,194	3,105,180	36,115	4/ 9/63	8/18/64	Peter Kiewit Sons' Co.
Del Valle and Sunol Pipelines	63-16	3,685,314	4,200,332	250,237	7/10/63	3/22/65	United States Steel Corporation
Second-Stage Pipeline from South Bay Pumping Plant to Dyer Canal	63-41	1,202,055	1,306,265	17,639	1/23/64	6/14/65	Green Construction Co. and Winston Bros. Co.
Niles and Santa Clara Division Pipeline	64-04	4,070,660	4,794,375	547,046	4/ 9/64	6/ 9/65	Kaiser Steel Corporation
South Bay Aqueduct Terminal Facilities	64-24	1,111,111	1,135,604	19,416	6/17/64	6/10/65	McGuire and Hester
Tunnel Extension Under Highway 50	65-49	88,696	92,875	1,002	1/ 4/66	5/18/66	Halbach and Flynn
Del Valle Branch Pipeline	67-67	803,210	808,295	7,758	1/11/68	4/16/69	M.G.M. Construction Co.
Modifications to South Bay Aqueduct	68-04	111,345	116,151	2,700	7/ 9/68	3/10/69	Power Construction, Inc.

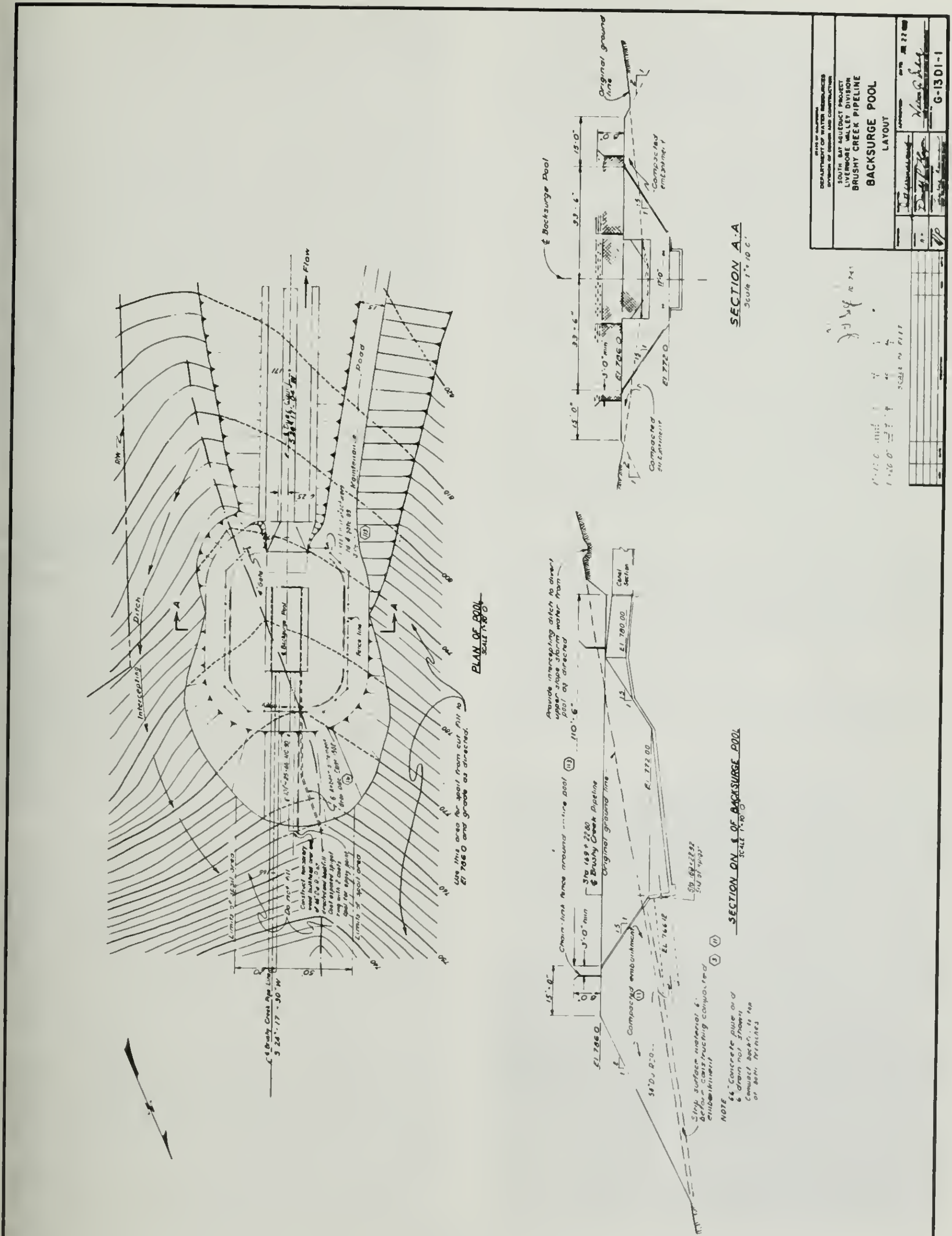

Figure 46. Dyer Back-Surge Pool Plon

Pipe, which was manufactured in 8- and 16-foot lengths, was lowered into the trench by side-boom tractors. Spigot and bell rings were wire-brushed clean and lubricated with vegetable soap, and a rubber gasket was placed on the spigot ring. A pipe section then was inserted into the previous length by aid of a come-along.

After a pipe section was set to line and grade, the joint was checked, a diaper was placed around the joint exterior, and backfill was placed about the pipe for support. Joints were filled with grout. After the grout had set, compacted or consolidated backfill was placed to 1½ feet above the invert. Consolidated backfill generally was used because of the limited work area and easier placement. Backfill was consolidated by simultaneous water jetting and vibrating and tested for acceptance prior to backfilling the rest of the trench. Interior joints then were filled with mortar.

The back-surge pool was constructed by the first-stage contractor. Concrete for the lining was hauled and placed with transit mix trucks. Six-inch waterstops were placed in contraction joints which then were sealed with Edoco sealant. A connection to the back-surge pool for the second-stage pipeline was installed and bulkheaded.

Vertical, 48-inch-diameter, concrete pipe was installed for access to the blowoff valves and 54-inch pipe for access to the air valves. At these access structures, consolidated backfill was extended to a drain rock base placed below the pipe to minimize settlement.

The second line was constructed in much the same manner as the first (Figure 47). Blasting techniques were adjusted and improved with the Department's blast-monitoring seismograph.

Figure 47. Installing Pipe for Second-Stage Brushy Creek Pipeline

Prestressed pipe was manufactured in 20-foot lengths with Carnegie-shape-steel-ring and bell rubber-gasketed joints. Because of favorable economics in procurement, prestressed pipe was used even where pressure heads did not require the increased strength of prestressed pipe.

The second line was pressure-tested by first draining the back-surge pool and removing the temporary, steel, dished bulkhead. With the bulkhead removed, the two lines formed a single interconnected system and, therefore, it was necessary to retest the first line and attribute any additional loss to the second line. The first line exhibited no measurable leakage. Water was introduced into the second line by controlled pumping and, for a week, the lining of the new pipe was allowed to absorb water, after which the canal beyond the back-surge pool was drained sufficiently to allow the water level in the back-surge pool and surge tank to equalize. A 24-hour leakage test revealed a loss of only 391 gallons or 2.4 gallons per inch of pipe diameter per mile for each 24 hours, well within acceptable limits.

Dyer Canal.

Design. This trapezoidal canal was designed for a capacity of 300 cfs on a hydraulic gradeline of 0.00041 at a velocity of 4.28 fps. A bottom width of 8 feet and side slopes of 1.5:1 were selected. An unreinforced, 3-inch, concrete lining was adopted.

Check structures were not contemplated in the original design; however, during construction, high uplift pressure from ground water threatened to overstress the canal lining. Therefore, four temporary check structures were added to ensure sufficient water depth in the Canal to balance the uplift forces. Checks were 3.5 feet in height and were formed by paving dikes with 3-inch mesh-reinforced concrete. A bond breaker was applied to the canal lining to facilitate later removal.

A siphon spillway with a capacity of 300 cfs was provided approximately 650 feet downstream from the back-surge pool to prevent overtopping of the canal embankment should the Canal become blocked downstream. The spillway discharges into a riprapped natural drainage channel modified to serve as a stilling basin. Siphon discharge is activated without full priming head by a splash plate located within the siphon barrel which creates a negative pressure zone in the siphon throat. This siphon spillway design also was used on Alameda Canal (Figure 34).

The transition to the 78-inch Altamont Pipeline is formed by a square-to-circular section within the last 12 feet of the structure. A trashrack and a 72-inch square slide gate were incorporated into the transition.

The invert slope of the Canal is relatively steep. Operation of the Canal demonstrated that overtopping at the lower end is possible. Also, severe drawdown would occur at the head of the Canal when shutting down. Therefore, a radial-gate check structure was provided at a location approximately one-third of the distance from the headworks, and a slide gate at the entrance to Altamont Pipeline was adapted for use as a check structure (Figure 48).

Construction. The Canal was excavated with a scraper to within 1 foot of invert elevation. Final grading was done with a trimming machine. In rock, the Canal was overexcavated 3 inches, and the rock was replaced with embankment material compacted by a sheepsfoot roller. Final grading then was done with the trimming machine. In areas of high ground water, the invert was overexcavated 4 inches and brought to invert grade with a layer of gravel drain material placed by a canal paving machine.

Concrete lining was placed with a self-propelled paving machine fed from a dual-drum paving mixer operating on the canal bank road. If the elevation of the road permitted, a conveyor belt transferred the mixed concrete; where the canal was far below the roadway, a crawler crane with bucket was used. The paver was supplied with concrete by dump trucks with four compartments, each holding 1⅓ cubic yards. Water was supplied by a trailing tank truck.

Elevation and alignment of the paving machine were controlled by feeler wheels preset to the subgrade. In areas of overexcavation, manual control was necessary. A finishing jumbo followed the paver, from which the flap-valve weep assemblies also were installed. The lining was sprayed with a curing compound.

As previously noted, a check structure was added to Dyer Canal after the Canal was in operation. Because of a tight water delivery schedule, shutdown of the Aqueduct was permitted for only two periods with maximum times of 12 and 5 days. The check structure contract also included removal of four temporary checks placed during the original contract.

During initial shutdown, the Canal was dewatered by pumping and three of the temporary checks removed. Minor alterations required at the transition section to Altamont Pipeline and the forming and placing of the check structure side walls and foundation slab concrete were accomplished during initial shutdown. Manufacturing delays in fabrication prevented gate installation during the second scheduled shutdown. Because of water delivery commitments, an additional shutdown could not be scheduled until six months later.

Upon resuming work, the Canal again was dewatered and the last temporary check removed. The sill plate and side plates for the gate were installed. The seals were attached to the gate which then was positioned and aligned, and the supporting members were grouted in place. Following the setting of the grouted parts, the gate operation was tested and, on refilling the Canal, it also was tested for water tightness.

Altamont Pipeline.

Design. The 2.3-mile Altamont Pipeline connects Dyer Canal to the Highway 50 tunnel. Altamont Canyon, in which the Pipeline is located, is narrow and the Pipeline, Altamont Pass county road, Altamont Creek channel, Western Pacific and Southern Pacific Railroad lines, and Pacific Gas and Electric and Pacific Telephone utility lines competed for the available space. Ground water, high-sulfate soils, and rocky ground also created problems. Several types of pipe were considered, including steel, concrete noncylinder, concrete cylinder, and modified prestressed concrete cylinder. Since long reaches of this pipeline would be subject to highway loadings and/or high earth covers, a steel pipe design would have required an excessive number of stiffener rings to control deflection and therefore was rejected. Nonprestressed cylinder pipe was preferred to modified prestressed pipe because of its greater rigidity, the external loading to be encountered, and the susceptibility of prestressing wires to damage by the sulfates in the soil. Therefore, nonprestressed cylinder and noncylinder concrete pipes were specified as alternatives, depending on the design head. For heads greater than 125 feet, cylinder pipe was required.

Altamont Pipeline lies almost completely in a formation of highly plastic fat clay. Near the western end of the Pipeline, the Greenville fault is crossed by the Pipeline in a tunnel under Highway 50. The Cierbo formation is encountered west of the fault on the eastern edge of Livermore Valley.

The Pipeline is in a tunnel at three locations under the Southern Pacific Railroad and at one location under the Western Pacific Railroad. The tunnels are supported by No. 7- or 8-gauge circular sections of liner plate. The annular space between the 72-inch-inside-diameter, mortar-lined and coated, steel, carrier pipe and liner plate was grout-filled. One-inch air release holes were required in the crown of the carrier pipe. The voids between the support liner and the excavated surfaces were required to be pressure-grouted (Figures 49 and 50).

The 573-foot-long tunnel beneath the Highway is supported with 93-inch-inside-diameter, No. 3-gauge, liner plate. The carrier pipe is 54-inch steel. To accommodate future widening of the Highway, the carrier pipe extends 140 feet and 186 feet beyond the east and west portals, respectively. The annular space about the liner also was grout-filled. However, to permit articulation of the carrier pipe in the fault zone, the carrier pipe was bedded 120 degrees on consolidated backfill. The remainder of the annular space along the fault zone was unfilled for inspection and access. A reinforced-concrete access structure was provided at each portal (Figure 51).

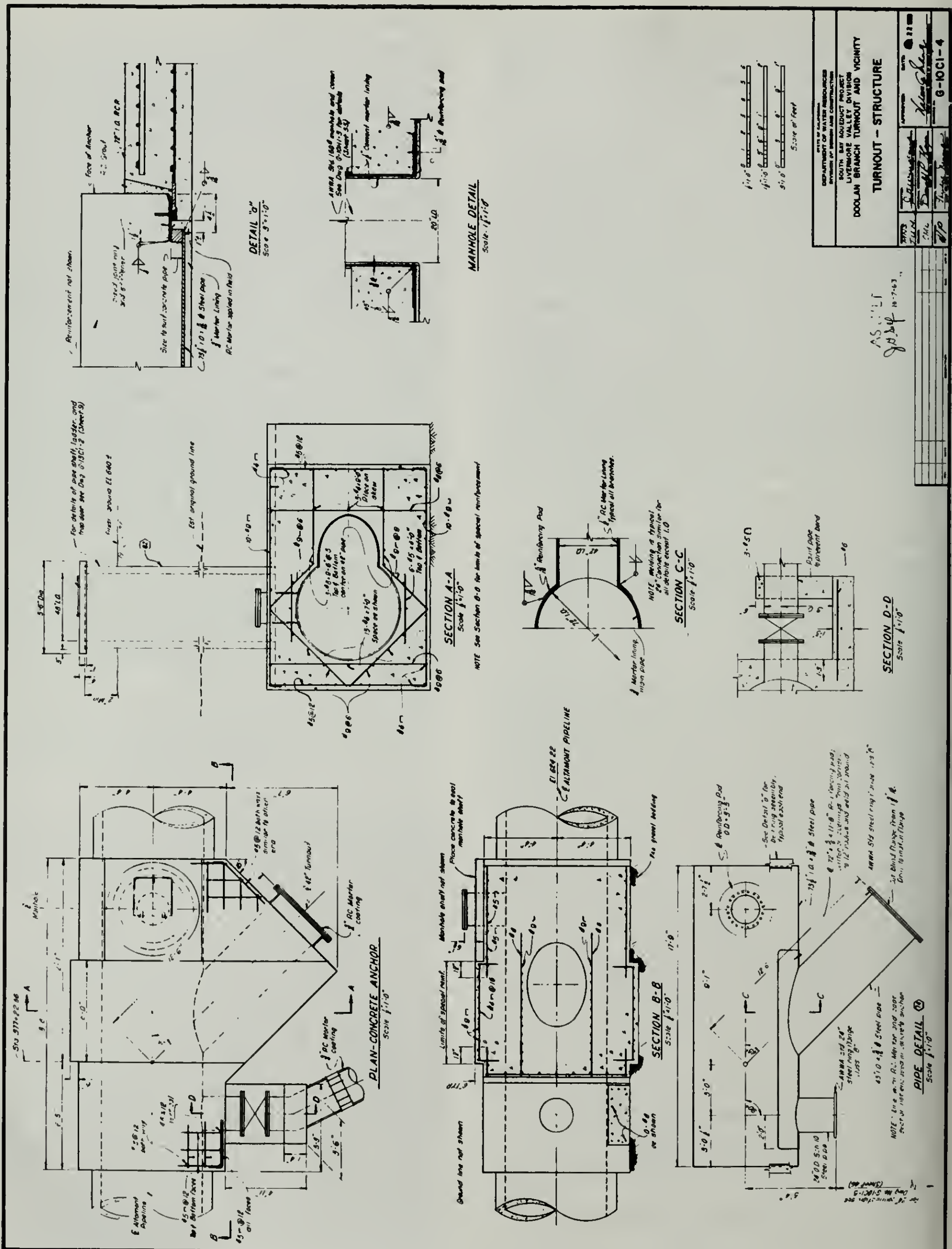

Figure 48. Doolan Branch Turnout

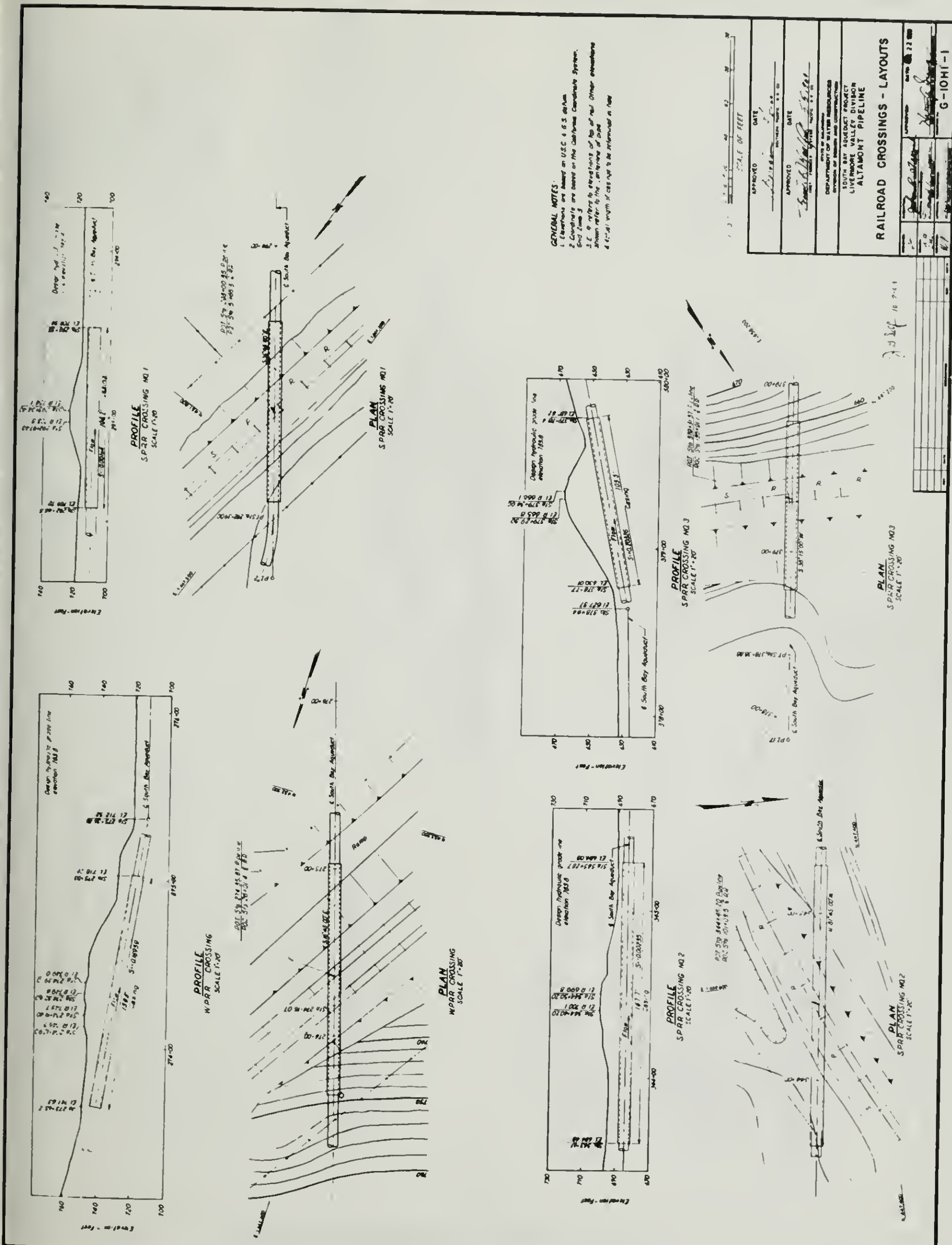

Figure 49. Plan of Tunnel Crossings at Railroads—Altamont Pipeline

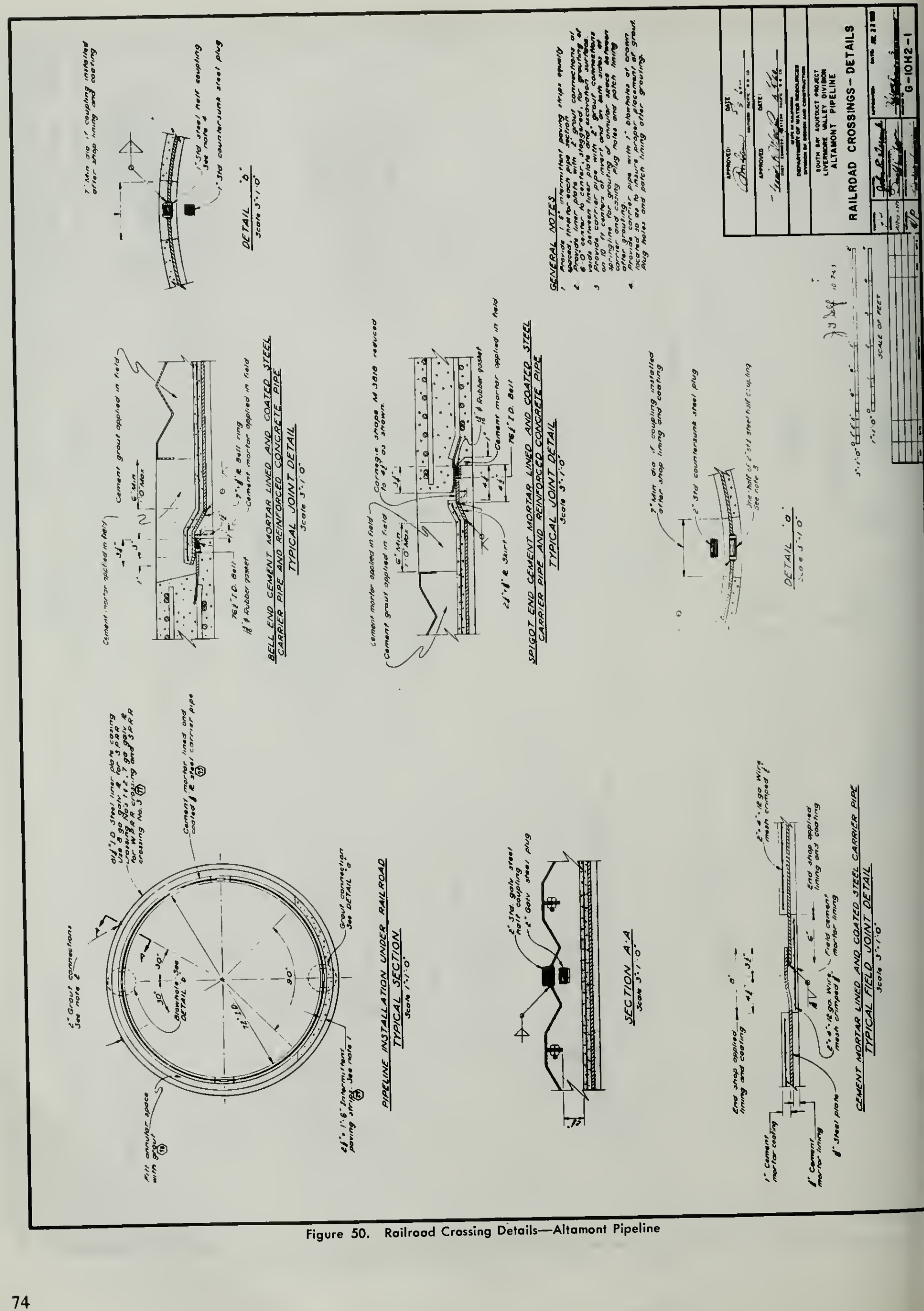

Figure 50. Railroad Crossing Details—Altamont Pipeline

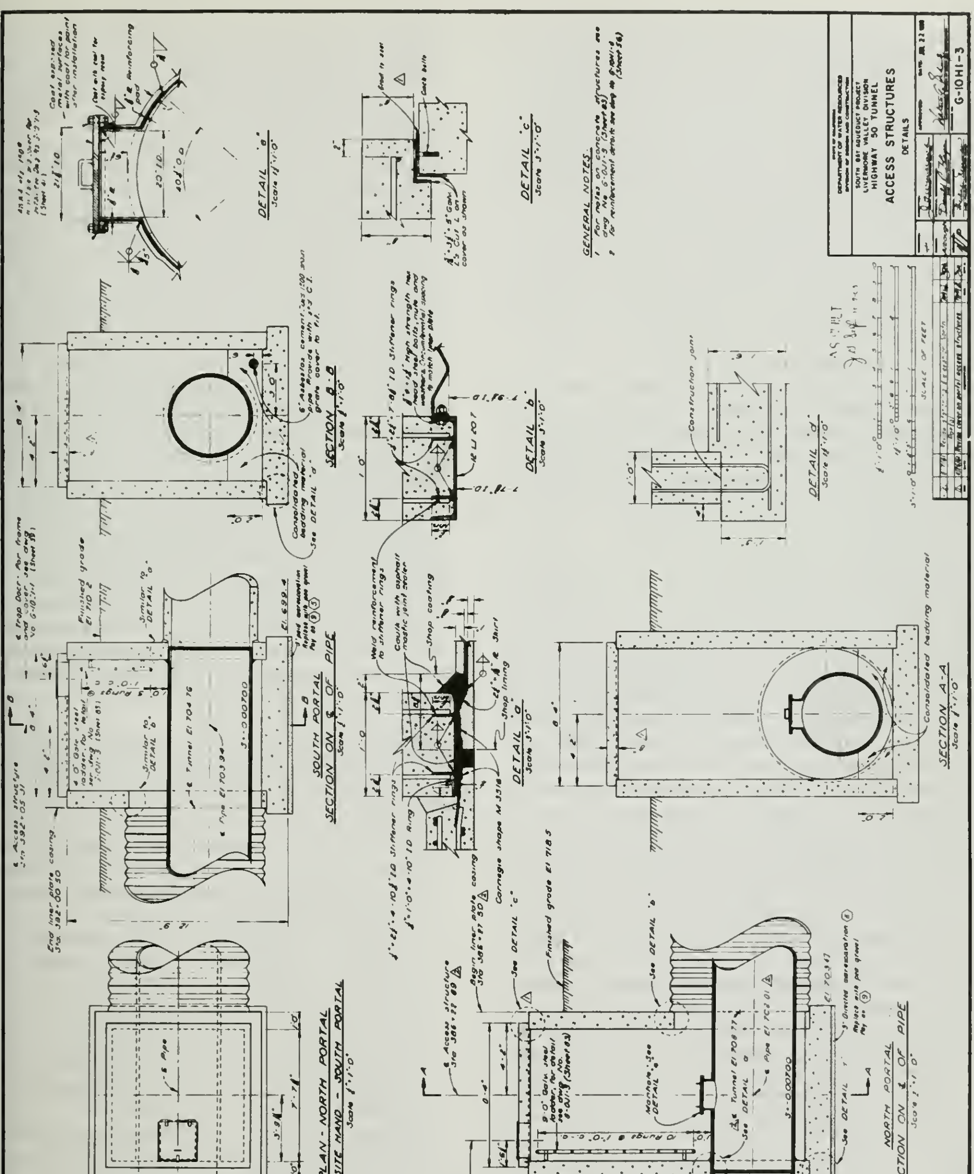

Figure 51. Highway 50 Tunnel Access Structure Details

Doolan Branch turnout and Altamont turnout are located just upstream of the Highway as separate, but closely spaced, wye connections. Both connections were designed for an internal pressure of 67 psi and are enclosed in a reinforced-concrete anchor block which acts as a rigid frame and carries the unsupported shell load. The stub for the future Doolan Branch branches from the main line at a 45-degree angle with a 12.5-foot section of 42-inch steel pipe closed by a 1⅞-inch-thick blind flange (Figure 48).

The 55-cfs-capacity Altamont turnout branches from the main line at right angles in a 24-inch-inside-diameter steel pipe. Flow is controlled by a 16-inch-diameter dispersion cone valve equipped with an impact energy dissipator. A 6-inch blowoff was incorporated in the dissipator structure.

Two vent structures and two 12-inch blowoffs are appurtenant to the main pipeline. The first section of the Pipeline is 78 inches to minimize blowback due to transition and slope change. The transition from Altamont Pipeline to Livermore Canal was designed as a 43-foot-long, reinforced-concrete, transition section. Restricted right of way required realignment of 1,600 feet of the Altamont Pass county road.

Construction. Pipe trench for most of the reach was excavated by dragline. In areas of ground water, the trench was overexcavated 10 inches and material replaced with gravel bedding. Normal sections of pipe were 20 feet in length. Wherever changes in alignment dictated, 16- and 8-foot sections were used. Pipe bedding and backfilling were similar to Brushy Creek Pipelines, although wet weather caused some delays.

Altamont Pipeline crosses by tunnel under four separate railroad interfaces. The lengths of these tunnels are 109 feet, 168 feet, 104 feet, and 140 feet. As the tunnels were excavated, 83-inch-diameter liner was installed and carried forward as the excavation progressed. The liner plate is of the two-flange type with each ring formed by six segments joined by bolted lap joints. Mortar-lined and coated carrier pipes extend through the completed tunnels. The 72-inch-diameter carrier pipe sections are 20 feet long with welded bell-and spigot-type joints. Pipe sections were pulled into the tunnels over curved concrete bolsters or saddles with a tractor winch, and final positioning was done with hydraulic jacks, wedges, and come-alongs. After welding the joints, the annular space between the liner and the carrier pipe was filled with grout.

Highway 50 was crossed in a 93-inch-diameter, grouted, liner-plate tunnel with a 54-inch, mortar-lined and coated, carrier pipe bedded in consolidated gravel. A total of 130,000 cubic yards of open-cut excavation was required for the portals.

The tunnel was driven entirely from the south portal. The first 100 feet were air-spaded without blasting while the remainder required only minor blasting. Liner plate was advanced close to the face and additional support was obtained by contact grouting. The first 213 feet of tunnel was grouted as it was driven while the remainder was grouted after holing through. Average grout take was 0.2 of a cubic yard per linear foot of tunnel. Carrier pipe sections were set to grade with chain and tackle, and the joints were butt-welded with back-up straps.

The Division of Highways requested and paid for a 210-foot westerly extension of the liner-plate section to accommodate future widening of the Highway. The Department of Water Resources prepared plans and specifications and awarded a contract for this extension which was completed in 1966. The extension was made in the previously excavated section at the west portal. Because the Pipeline was in operation while this work was underway, a 42-inch-diameter, bypass, steel pipeline had been installed previously and extended to the Livermore Canal section.

After removal of the existing pipe and the south portal access structure, the trench was widened to accept the liner-plate sections. The lower half of the liner segments then were erected, and consolidated bedding for the pipe was placed followed by the first lift of consolidated backfill about the lower half of the liner plate. Steel carrier pipe then was laid, joints welded, bypass line disconnected, and the new line placed in operation. Backfill for the pipe liner was brought up simultaneously and consolidated by jetting and vibrating. After the backfill was within 1 foot of springline, the liner-plate crown sections were installed, and consolidated backfill was completed. A new access structure was constructed similar to the one removed.

Livermore Valley Canal.

Design. The 2.2-mile Livermore Valley Canal is similar in design to Dyer Canal and has a capacity of 300 cfs. The entire length of Livermore Valley Canal is in cut section in the Cierbo formation, alluvium, or clayey soil.

Livermore Valley Canal was used for testing different lining thicknesses and underlining membranes. The following briefly describes the test sections.

Total Length (feet)	*Lining Thickness (inches)*	*Earth Treatment Under Lining*
904	3½	None
500	3½	Plastic film—lapped joints
500	3½	Plastic film—sealed joints
500	3½	Asphalt membrane
500	3½	None
4,300	3	Special joint material
300	2½	None
500	2½	Plastic film—lapped joints
500	2½	Plastic film—sealed joints
500	2½	Asphalt
500	2½	None
625	3	None

The tests attempted to develop empirical relationships between flow capacity, side slopes, depth of flow, and uplift. However, the data did not correlate well enough to establish specific relationships.

Different sublinings were tested and observed in an attempt to develop relationships between seepage and random cracking. An asphalt film, $\frac{3}{16}$ of an inch thick, was applied by spraying; however, unless the asphalt was applied immediately before the concrete lining, the membrane tended to slough. Plastic film was 12 mils thick and, unless the water content of the concrete lining was controlled carefully, the concrete had a tendency to slide on this smooth sublining. Moisture blocks were placed below the lining at various points to monitor the seepage rate for the different types of sublining. As a result of the sublining test installation, it was concluded that any benefit realized by reduction in seepage due to sublining would be outweighed by the higher costs of installation and maintenance.

A two-radial-gate check structure was incorporated in the design just upstream from the end of Livermore Valley Canal to provide operational flexibility.

Cross-drainage structures were provided as needed and included four 24-inch culverts, three 24-inch overchutes, and six 18-inch drain inlets. Three one-way, 20-foot-wide, timber-deck, road bridges were included in the contract.

Construction. The Canal was excavated and embankment constructed similar to Dyer Canal. Overexcavation to facilitate fine grading also was similar; however, ground water and rocky ground were encountered.

Asphaltic membrane test sections were installed by transporting the asphaltic material to the site in insulated, 22-ton, tank trucks and pressure applying it to the moistened subgrade with hand-held spray nozzles to a thickness of $\frac{3}{16}$ of an inch. Concrete lining was placed immediately afterward.

One test section was used to investigate a plastic waterstop material called Constop which was installed in both longitudinal and transverse joints. Longitudinal joints were formed by threading the waterstop material into tubes which ran under the paver screed and guided the waterstop into proper position. After the concrete was in place, transverse joints were formed from the finishing jumbo by forcing the Constop into the concrete while it was still plastic. The surface was finished with hand-held vibrating floats. Many core samples were taken to observe the consolidation of concrete around the Constop and note its position. In general, longitudinal joints appeared satisfactory while the methods of placing the transverse joints needed to be improved due to poor consolidation of the lining around the waterstop particularly at its crossing with the longitudinal joint.

Patterson Reservoir

Design. Patterson Reservoir is a 100-acre-foot-capacity reservoir at the end of the Livermore Valley Canal reach of South Bay Aqueduct which provides emergency water storage for Alameda County Flood Control and Water Conservation District's Zone 7 Treatment Plant.

The Reservoir is formed by a compacted earth embankment adjacent to the Canal. Crest elevation of the perimeter embankment was set at 712.50 feet and the canal-side embankment at 708.72 feet. Average depth of the Reservoir is 29 feet. Inboard side slopes are 2:1 and the crest is 15 feet wide. The reservoir invert is sloped for complete drainage of the Reservoir through a 12-inch-diameter, reinforced-concrete, drain line. The drain line discharges into a reinforced-concrete, impact, energy dissipator. The drain is controlled with a 12-inch plug valve.

Aqueduct flows enter the Reservoir over a 175-foot-long concrete-lined weir on the right side of Livermore Valley Canal. The weir lining extends down the reservoir inboard slope for 6 feet. The reservoir lining along the Canal is formed by (1) a 3-inch surface layer of asphaltic concrete; (2) an 18-inch layer of compacted impervious soil; and (3) an underlying, 15-inch, sand-drain blanket. A 6-inch open-tile drain is located beneath the sand blanket to convey percolating waters collected by the blanket drain from the uphill sand lenses of the Canal to the 12-inch drain line (Figure 52).

Water deliveries are made from the Reservoir through a 42-inch outlet located at the southerly corner of the Reservoir to the nearby Zone 7 Treatment Plant. Water also can be delivered through a 30-inch, reinforced-concrete, bypass pipeline directly from the Canal to the Treatment Plant.

Construction. The embankment was placed in the same manner as the canal embankment. The specifications required an overfill of 12 inches for the impervious material. As the material was placed in horizontal layers for a full scraper's width down the embankment slopes, there was considerable excess overfill which was removed. The impervious material on the weir embankment slope was increased to 21 inches with the underlying filter blanket reduced to 12 inches. Due to drying and cracking of the impervious material, slope paving was changed from asphaltic concrete to a 3-inch layer of reinforced concrete. The joint between the weir keyway and the slope paving was filled with dehydrated cork and sealed with grout.

The asphaltic concrete on the bottom of the Reservoir was placed with a paver and rolled with an 8-ton steel-wheeled roller. The sloping sides were paved in one pass from a hopper-fed slip form paver which was drawn up the slope by a side-boom tractor. The paving was compacted with a roller similarly drawn up the slope. All surfaces were sprayed with an asphaltic primer prior to paving. The crest road was paved between concrete curbs with 3 inches of Type A asphaltic concrete, with one pass of a paver, and then compacted with a steel-wheeled roller. A Portland cement seal coat, consisting of a slurry of cement, water,

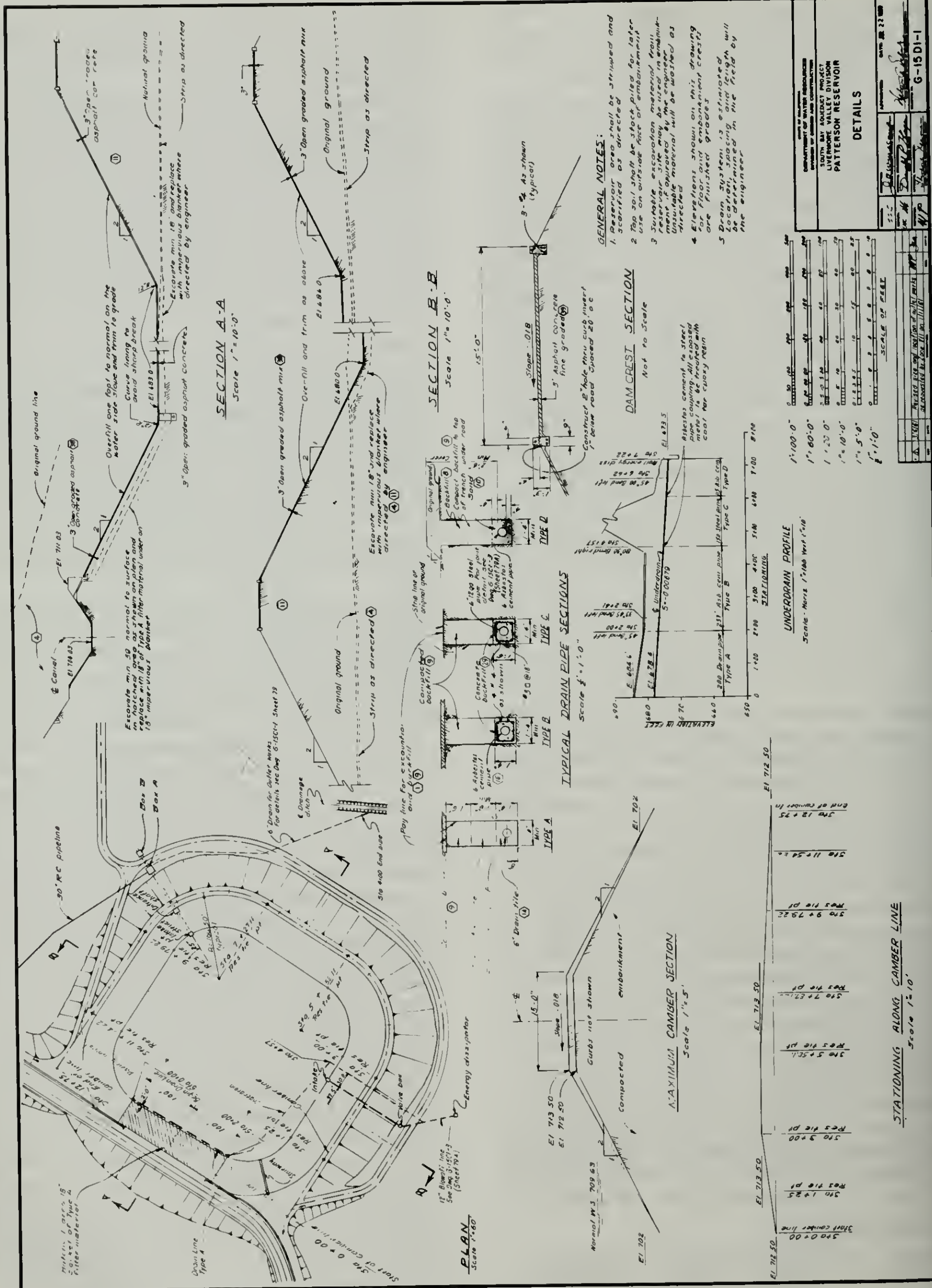

Figure 52. Potterson Reservoir—Plan and Sections

and calcium chloride, was sprayed on the surface.

Completion of Patterson Reservoir (Figure 53) concluded the Surge Tank to Patterson Reservoir contract. A dedication ceremony on May 10, 1962, attended by Governor of California Edmund G. Brown, at the Patterson Bypass turnout celebrated the initial delivery of water through South Bay Aqueduct.

On the initial filling of the Reservoir, the keyway between the weir section and the embankment apron leaked. Subsequent operations aggravated the condition and the keyway joint opened 3 inches. Excessive flows in the tile drain line indicated that voids also had formed beneath the apron slab and, subsequently, a contract was awarded for repairs.

The extent of the voids was determined by drilling 3-inch inspection holes in the apron. Voids were filled by injecting a dry mixture of sand and 6% cement, after which the holes in the apron were plugged with concrete. A row of flap-valve weep assemblies were placed 7 feet apart in a row above the drain line. The bottoms of the weep valves penetrated into the impervious layer and were set in a pocket of sand. The surface around the valves was painted with an epoxy, the keyway joint was cleaned of cork material and sandblasted, and all voids were filled with dry-pack mortar and sealed with a polysulfide compound. The junction of the weir and the slab then was capped with cement mortar and sprayed with a membrane curing compound. Forty-five 14-inch-long No. 6 dowels were grouted into 2-inch holes tying the weir section to the apron. During the investigation leading to the repair contract, inspection holes were drilled at each end of the tile drain. These holes were cased with 4-inch pipe and left in place for use by the repair contractor. Later, it was discovered these holes had broken the tile and sand had plugged the drain. Also, percolating water around these pipes had developed bell-shaped cavities 3 to 4 feet in diameter. The repair contractor flushed the drain line, filled the cavities with cement grout, and plugged the top of the drilled holes with concrete.

Figure 53. Completed Patterson Reservoir

After the repair, drawdown of the Reservoir again revealed extensive voids in the vicinity of the plugged holes at the tile drain. The concrete plugs probably had shrunk and piping had occurred. Examination of the canal lining below the weir section indicated sufficient cracks to permit water to flow below the reservoir slab. It was decided to replace the existing slab below the weir section with asphaltic concrete, increase the thickness of the impervious layer to 30 inches, and provide a better connection between the surface slab and the weir. The tile drain and the cracks in the canal lining also were scheduled for repair under the modification contract.

During construction, it was necessary to maintain canal operations and water deliveries. Therefore, during a short shutdown period, earth dams were placed in the canal above and below the Reservoir, and a temporary 24-inch steel line was installed through the earth plugs to provide downstream deliveries. A temporary timber bridge with one pier in the canal prism also was constructed to provide access to the impervious material borrow area across the canal. Extended periods of rainy weather interfered with construction, particularly in placing the impervious material and repairing the canal lining. The existing concrete embankment apron was broken up and removed. The old keyway and a section of the weir slab also were removed. A concrete curb then was placed to form a junction between the weir slab and the new asphalt apron. The face of the curb was painted with epoxy just prior to placing the new lining. Canal lining cracks were repaired by chipping, cleaning, and sealing with a polysulfide sealant. The tile drain was removed, cleaned, and relaid on a 16-inch layer of bedding material. Building paper was placed over the upper half of the pipe and then covered with a 9-inch layer of pea gravel and a 9-inch layer of sand. This upper sand blanket provided contact with the sand layer between the increased thickness of the impervious layer and the asphaltic concrete pavement. The reservoir drain line also was modified to provide above-water operation of the valve. Since completion of the new apron, the Reservoir has operated satisfactorily.

Alameda Canal.

Design. The 6.9-mile Alameda Canal is the final and longest reach of canal in the South Bay Aqueduct system. The Canal is trapezoidal in section, with a bottom width of 8 feet and side slopes of 1.5:1, and is lined with 2½ inches of unreinforced concrete to a depth of 7 feet. Capacity is 300 cfs at a depth of 5.58 feet and a velocity of 3.28 fps. The first 10% of the canal length is in the Cierbo formation, 60% is in the Livermore formation, and the remainder is in alluvium and terrace gravels. The alignment crosses traces of the Telsa, Carnegie, and Corral Hollow faults.

Figure 54. Automatic Trashrack Cleaner at Del Valle Check

Figure 55. Trimming of Canal Prism—Alameda Canal

Figure 56. Canal Paving Machine—Alameda Canal

The Livermore formation consists of slightly consolidated, interbedded, lenticular gravels and cobbles; silty sandy clays; gravelly clays; and plastic clays. This formation is poorly cemented and is softer than the sandstones of the Cierbo formation. The formation generally contains sufficient fines and intermediate grain sizes to make the beds essentially impervious. Alluvium consists of silty and sandy clays of medium-to-high plasticity, while terrace gravels generally are clayey gravels. Boulders occasionally occur in both units.

This reach of aqueduct contains four siphons, two of which incorporate single-gated check structures. Two of these siphons are stream undercrossings with blowoffs; one is a road undercrossing; and the fourth is an 880-foot-long, combined road and stream undercrossing. A separate check structure contains two gates for flow control. There are two county road bridge overcrossings, five timber farm road bridges, one sand trap, and numerous cross-drainage structures including two pipe and one box overchutes (Figures 36 and 37). The transition from the canal prism to Del Valle Pipeline also contains a single radial gate.

A 300-cfs siphon spillway is located slightly upstream from the transition. It provides a wasteway for canal flows should a blockage or other emergency condition occur. The spillway can be activated by closing the Del Valle check gate. The Del Valle check structure originally was equipped with a conventional trashrack. However, due to an unanticipated algae growth problem in the Canal, it was later necessary to install an automatic trashrack cleaner at this location (Figure 54).

All siphons were designed for the same size and type of pipe as it was felt bidding economics would result. Based on a head-loss versus pipe-size study, 84-inch-inside-diameter, noncylinder, reinforced-concrete pipe was selected. Steel pipe was not considered because of the low heads involved and the more expensive cathodic protection.

The county road crossings are single-span, reinforced-concrete, T-beam structures supported on abutments with the footings belled out for increased bearing area. The pipe overchutes are clear span crossings, two of 20-inch pipe and one of 24-inch pipe. These are supported on each bank with a combined concrete anchor and support block. The 4- by 6-foot box overchute is supported on two piers 25 feet on each side of the canal centerline. The outlet of the box contains a dissipating structure with four 9- by 15-inch roof blocks and three similar floor blocks (Figures 36 and 37).

Construction. The Canal was excavated to within 2 feet of canal invert by dozers with hydraulic rippers and by push-loaded scrapers. A dragline was used to extend the rough excavation to within 6 inches of invert. The remainder of the material was removed with one pass of a trimming machine (Figure 55). Canal embankment was placed with scrapers and compacted with 5-foot by 5-foot, double-drum, sheepsfoot rollers. Approximately 2,000 lineal feet of overexcavation was required because of ground water or rocky formation. The western limit of the contract was extended 500 feet to provide a transition to Del Valle Pipeline. This extension passed under a high-pressure gas line and required considerable hand labor.

The canal lining was placed with a self-propelled mixer and paving machine followed by a finishing jumbo. Concrete then was sprayed with a curing compound (Figures 56, 57, and 58). Alignment and grade of the paver and trimmer were maintained by microswitches acting on the crawler tracks from guide wires set on each berm of the canal prism. The controls were sensitive to 0.02 of a foot (Figure 59). Overexcavation was replaced with concrete. Some tearing of the concrete occurred while cutting the transverse contraction grooves with a knifelike blade operated from the paver. Because the maximum-size aggregate was 1¼ inches and the lining thickness only 2½ inches, considerable hand finishing was required.

A new method was developed in placing the weep valve assemblies. In previous canal sections, it had been difficult to relocate the filter pockets after the lining was placed. Probing was slow and inaccurate, and wires protruding from the filter material usually were displaced or bent by the lining operation. These difficulties were overcome by placing the filter material, a ¾-inch gravel, in a burlap sack which was placed in designated excavations in the subgrade. A spring steel rod protruded through each sack and would pop through the concrete after passage of the paver. A steel template in the shape of the valve then was used to form the required pocket in the fresh concrete for receiving the weep valve assembly. This method proved satisfactory.

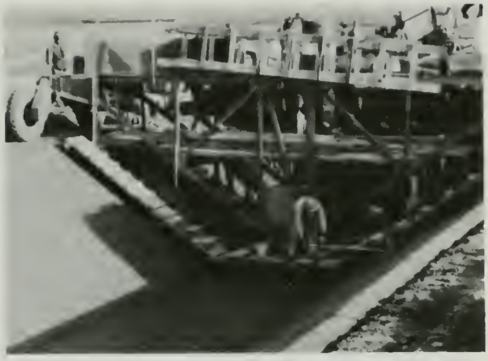

Figure 58. Finishing Jumbo for Canal Lining—Alameda Canal

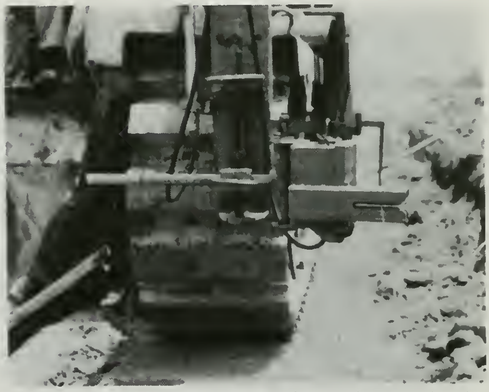

Figure 59. Electric Grade-Control Switch on Guide Wire—Alameda Canal

Figure 57. Self-Propelled Mixing Machine for Canal Lining—Alameda Canal

Figure 60. Inlet of Arroyo Seco Check Siphon

Figure 61. Tesla Road Siphon Inlet

Figure 62. Forming Arroyo Seco Siphon

Four road crossings and stream crossings were provided by inverted siphons complete with warped wingwall transitions (Figures 60, 61, and 62). Eighty-four-inch-inside-diameter concrete pipe was installed at each siphon crossing. Two reinforced-concrete bridges were constructed: one at Lupin Way (Figure 63) and one at Greenville Road (Figure 64).

Cross drainage was provided by reinforced-concrete pipe, culverts, concrete box culverts, or reinforced-concrete overchutes.

On August 30, 1966, a slide occurred on Alameda Canal at Station 622+50 (Figure 65) while the Aqueduct was in service. The hillside above slid into the Canal, displacing the concrete lining from left to right. However, the compacted enbankment on the right side of the Canal did not fail and no water was released from the Canal.

Repairs were started immediately with two dozers, a dragline, a front-end loader, and two dump trucks to remove the slide material. Starting approximately 2 feet below, the canal invert was restored with material compacted with a sheepsfoot roller. A grader trimmed the compacted embankment to grade (Figure 66), after which pneumatically applied concrete reinforced with wire mesh was placed (Figure 67). The Aqueduct was back in service on September 6, 1966.

Del Valle Pipeline Through Mission Tunnel

Del Valle and Sunol Pipelines and La Costa and Mission Tunnels form a pressure conveyance system to Santa Clara Pipeline. Del Valle Branch Pipeline connects Del Valle Pipeline to the off-line Del Valle Pumping Plant, which is part of the Lake Del Valle pumped-storage feature.

Del Valle and Sunol Pipelines.

Design. The check structure at the transition to Del Valle Pipeline is the last in-line control in the South Bay Aqueduct. Therefore, all water beyond that point must be delivered through turnouts, stored

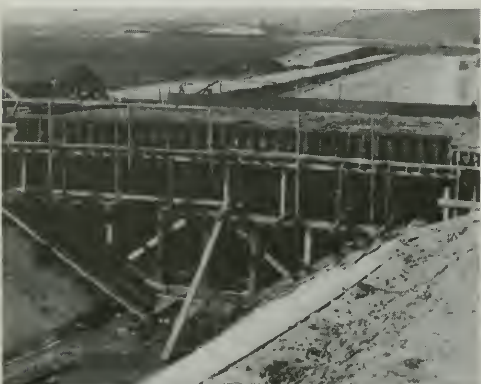

Figure 63. Lupin Way Bridge Under Construction

Figure 64. Completed Greenville Road Bridge

in the system, or wasted.

Del Valle Pipeline and all but the last mile of Sunol Pipeline are located in the Livermore formation, which consists of poorly cemented clays, silts, sands and gravels, and recent alluvium. The final mile of Sunol Pipeline is in some of the younger rocks of the Diablo Range and in the Claremount, Oursan, Tice, and Hambre formations of the Tertiary Monterey group. The younger formations consist of shales, siltstones, and sandstones with many folds and faults. The more prominent faults crossed by the Pipelines are McGuire Peaks, Calaveras, and Stonybrook. The Calaveras fault is considered active and was discussed earlier in this chapter. Ground water is more prevalent along this portion of the Aqueduct.

Preliminary design for this reach contemplated that the primary method of conveyance would be by canal. However, to avoid landslides common in the area, provide a more direct route, reduce excavation, and avoid high canal embankments, a pipeline concept was adopted in the final design (Figure 68).

Specifications were prepared for steel- or concrete-pipe alternatives and bids were invited on two schedules. Separate excavation bid items were classified on the basis of relative difficulty of excavation.

Concrete-pipe specifications were similar to those for the Altamont Pipeline, except prestressed-concrete cylinder pipe was added as an alternative. Steel-pipe specifications required mortar lining, field- or shop-applied, and a coal-tar coating. Earth cover over the steel pipe was limited to 10 feet. At the Mission Tunnel approach, where the cover reached 30 feet, concrete pipe was required. Inside diameters for either type of pipe were 84 and 90 inches, air relief valves and vents were incorporated in the design, and minimum wall thickness of the steel pipe was set at 0.5% of the diameter. However, because of the shop-applied mortar lining alternative, deflection was limited to 1.5% of the diameter. Because backfill and bedding conditions are critical in minimizing cracking of the lining in the case of steel pipe, the specifications required either rocky or wet ground to be overexcavated 9 inches and replaced with compacted backfill, or the pipe was to be bedded on 2 inches of loose granular material. Proper bedding and backfill are particularly important where steel pipe does not flow full and, thus, internal pressure is unavailable to help resist the weight of the backfill. Three bids were received for steel pipe and four for concrete pipe. The two lowest bids were for both schedules on the steel-pipe alternative.

As designed, the steel pipe is either an 84-inch or 90-inch inside diameter with varying plate thickness, based on capacity, external loading, and internal pressure.

Short cut-and-cover lengths of concrete pipe were specified at the portals of La Costa Tunnel and the east portal of Mission Tunnel to permit connections to be made to the cast-in-place portal sections (Figure 69).

Figure 65. Alameda Canal Slide

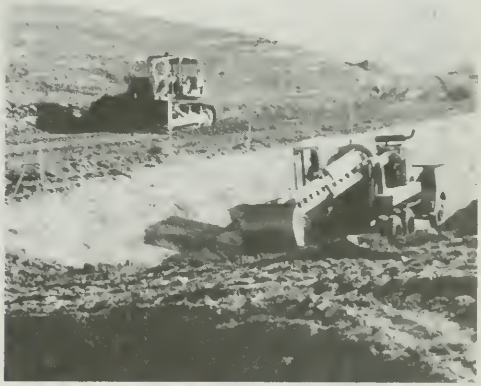

Figure 66. Trimming Compacted Embankment—Alameda Canal

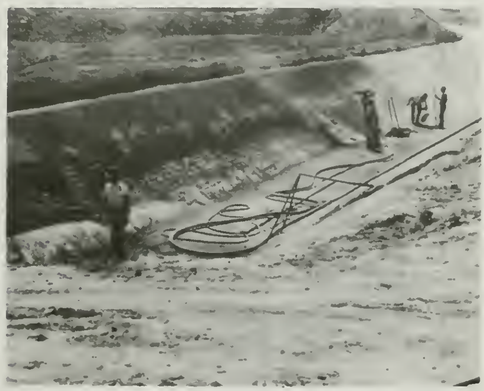

Figure 67. Application of Pneumatically Applied Concrete Lining—Alameda Canal Repair

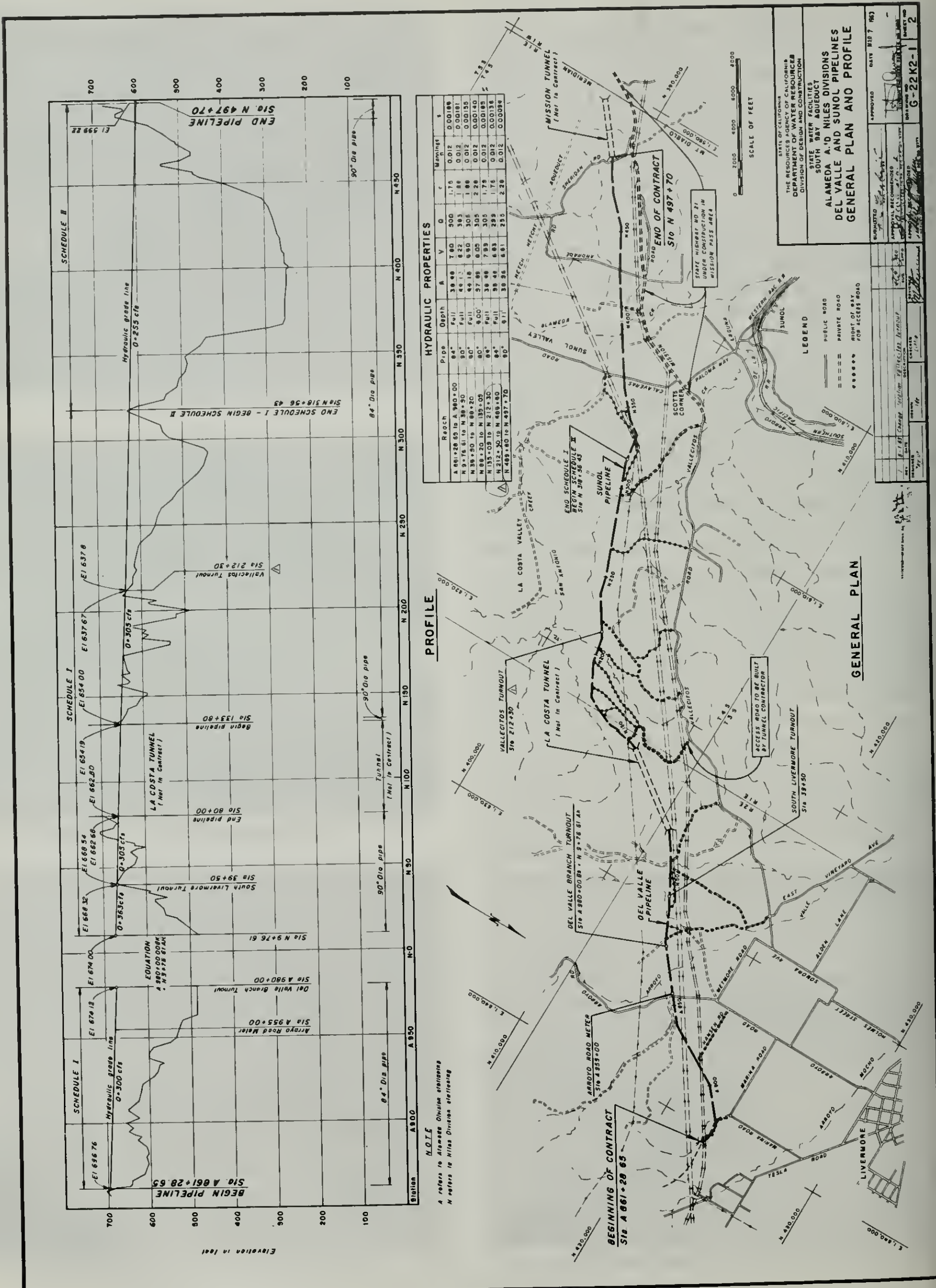

Figure 68. Del Valle and Sunol Pipelines—Plon and Profile

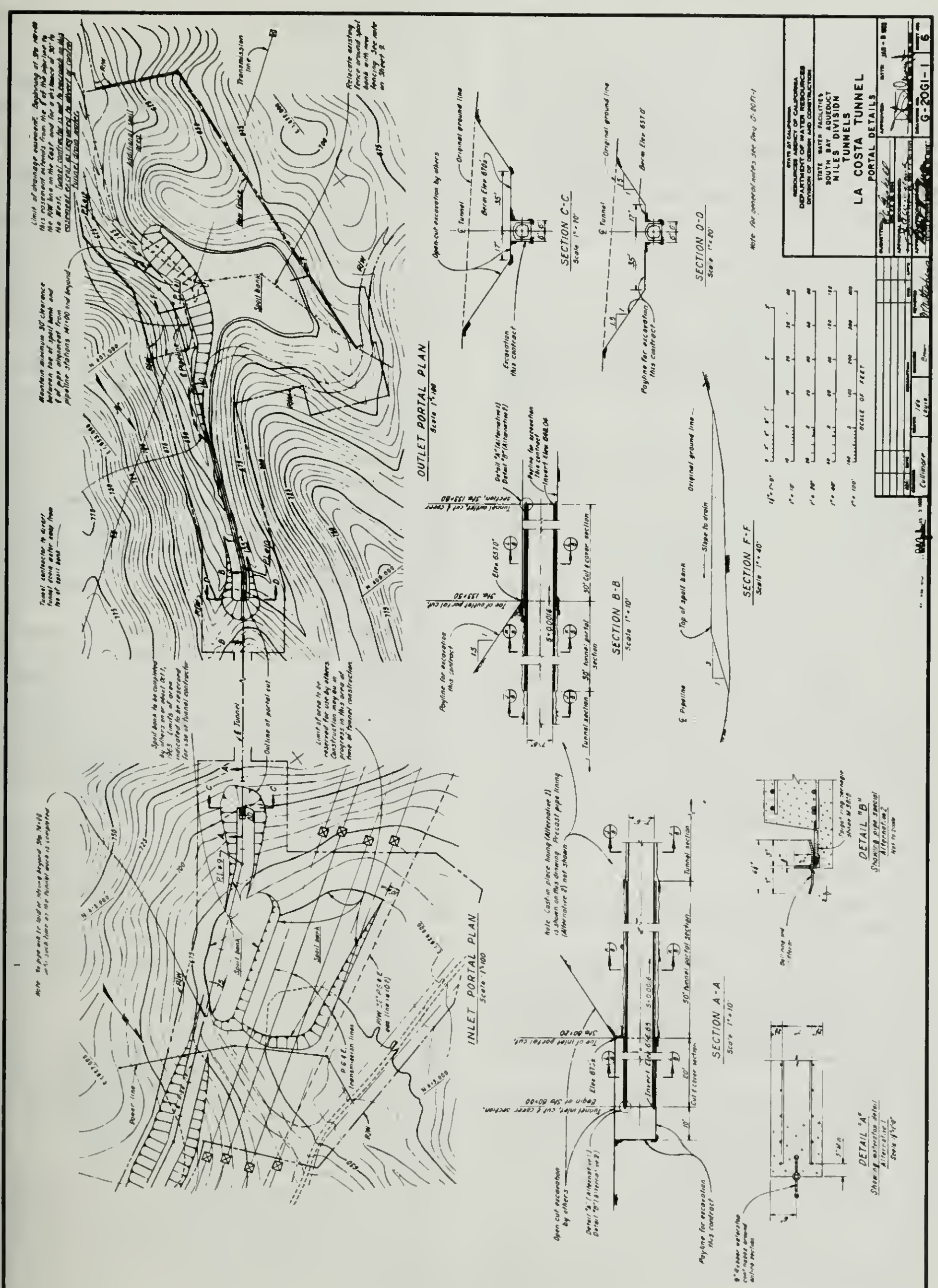

Figure 69. La Costa Tunnel Portal Details

Concrete riser pipes are used for air valves, blowoffs, vents, and standpipes supported on reinforced-concrete pads placed on top of the main pipe. Vent pipes and the main pipe are encased in reinforced-concrete anchor blocks at their juncture (Figure 70).

Halfway across Arroyo Del Valle, a tee connection was provided to Del Valle Pumping Plant, Del Valle Dam, and Lake Del Valle. This 90- to 60-inch tee has a saddle-type reinforcing plate and is further reinforced by stiffener plates welded to the top and bottom of the connection. The entire assembly is encased in a reinforced-concrete anchor block.

The South Livermore turnout, about midway between the Del Valle Branch connection and La Costa Tunnel, is a 90-degree wye sized for 57 cfs through a 48-inch pipe. The wye is similar to the Del Valle connection, except stiffeners were not required for the saddle reinforcement nor was reinforcement required for the anchor block.

The Vallecitos turnout on Sunol Pipeline, 1½ miles downstream from La Costa Tunnel, is the most complex structure in the Pipeline. The water user requested maximum potential head for delivery to his system. Therefore, the turnout was located at a summit on the Pipeline and designed for a capacity of 120 cfs with no flow beyond the summit. During construction, the turnout was redesigned for additional service as an emergency spillway when downstream flows are rejected. Because the hydraulic gradeline is close to the elevation of the Pipeline, the turnout structure is open at the top and covered with a five-section, galvanized, steel grating. The structure is a rectangular reinforced-concrete box operating as a drop inlet with a 7⅓-foot difference in invert elevation of the main pipeline and the 42-inch, steel, turnout pipe. This elevation is equivalent to the head required to divert 120 cfs with no flow over the summit of the main pipeline. Overflow relief as a spillway is provided by a 2.75-foot by 9-foot notch weir in the north wall of the structure (Figures 71 and 72).

Construction. The prime contractor fabricated the steel pipe and subcontracted all of the other contract items.

Trench excavation was divided into three zones: Zone I was the largest, covering gently rolling and level areas; Zone II was in steep slopes from the outlet portal of La Costa Tunnel to Calaveras Road; and Zone III was in the beds of Arroyo Del Valle and Alameda Creek.

The alignment was pioneered with dozers and scrapers. The pipe trench was excavated with a dragline and a 1½-cubic-yard backhoe (Figure 73). A front-end loader was used for fine grading. A small backhoe was used to excavate bell holes, and the material was removed by a clamshell.

Ground water made excavation difficult. In some locations, it was necessary to dewater by pumping. Areas of high ground water were overexcavated and backfilled with compacted bedding material. In some cases, drain rock was placed under the bedding material:

Steel pipe in 40- or 80-foot lengths with welded joints and shop-applied coal-tar enamel was used. Mortar lining was field-applied (Figure 74).

Special excavation was required in the vicinity of Alameda Creek because of the presence of ground water and aggregate plant sediment ponds. Deposits from this aggregate plant consisted of layers of damp or saturated silk separated by thin layers of saturated, soft, fat clay. An existing, high-pressure, gas line across this area was buried in gravels, and the consolidated backfill for that line was used as a stabilizing bank for the right side of the Sunol Pipeline trench excavation.

The design contemplated removal of sediments down to the underlying stream gravels and construction of a compacted embankment in which the pipeline trench would be excavated. Several slides in the excavation forced a change in construction methods. The sediments were cut back to reduce the surcharge, the trench was widened, and the left slope was flattened. Increased ground water pumping and placing the compacted backfill soon after excavation also helped. As soon as the compacted backfill was finished, the pipe trench was excavated and the pipe laid and backfilled.

In laying a section of pipe (Figure 75), the required insert was marked on the spigot end of the previously laid section. The section then was lowered into the trench at a 15-degree angle with the previously placed section, and the pipe joint was made (Figure 76). After the section was laid and set to line and grade, tack welds were made to secure the added section. About every 400 feet, a joint was temporarily left free as an expansion joint.

Initially, field welding used Linde's short arc, semiautomatic, gas-shielded, welding process. This method proved unsatisfactory as normal, outdoor, air currents would disperse the argon-carbondioxide gas shield surrounding the arc, resulting in blow holes in the welds. Later, an electric arc-welding process was used with satisfactory results. Welds were checked by radiographing. The welded joints were field-wrapped, inspected, and holidays repaired.

Satisfactory consolidation of backfill was a persistent and perplexing problem. Finally, after several variations had been tried, the first two lifts were accomplished with two jets and vibrators applied on each side of the pipe, and the third lift then was compacted with a hand-held compactor.

Consolidated backfill was tested and areas of failure were reworked and retested. Test pits were dug at approximately 500-foot intervals to verify that the consolidated backfill was fully supporting the pipe; if voids were discovered, the backfill was removed and replaced.

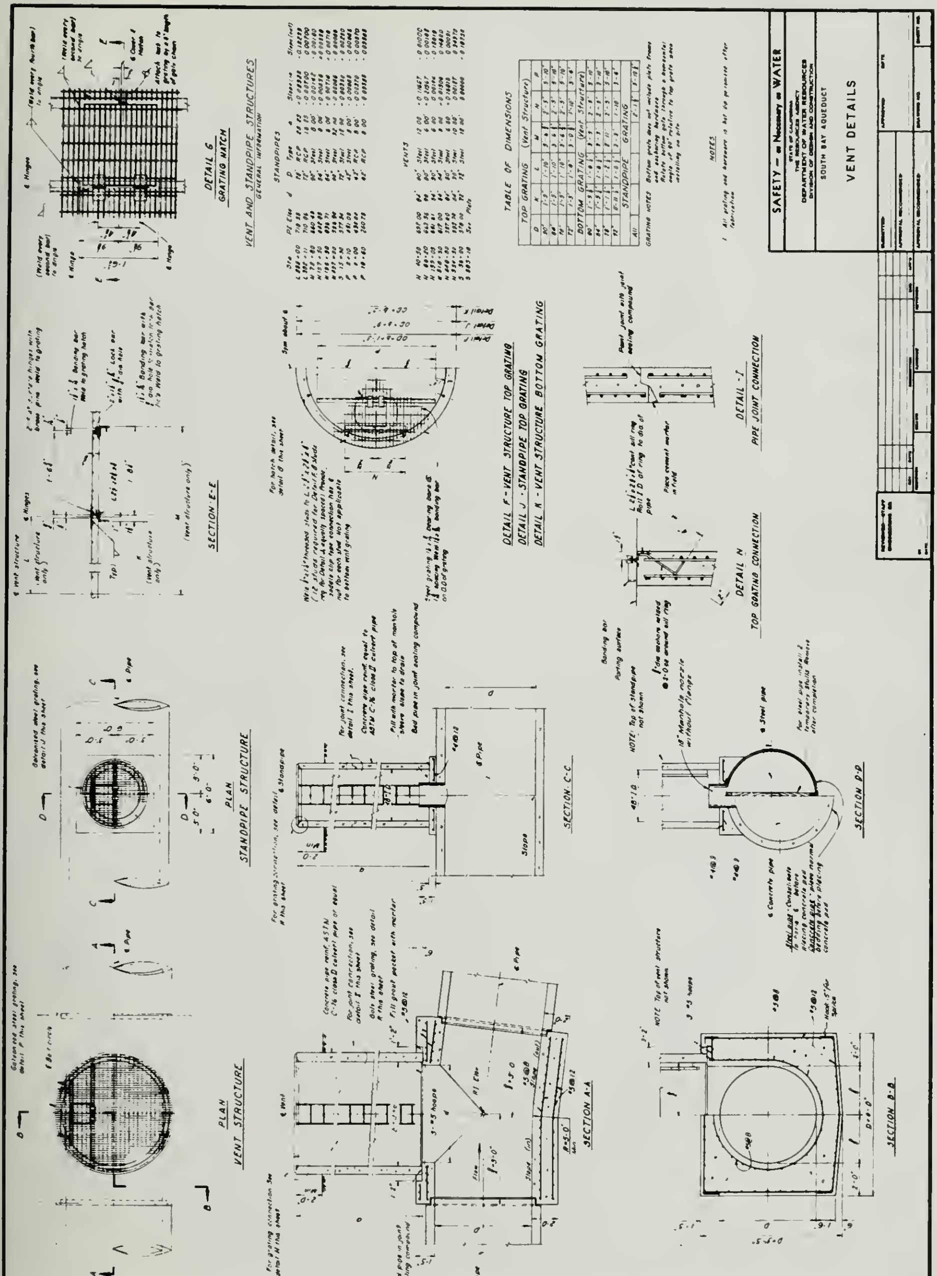

Figure 70. Typical Vent Details

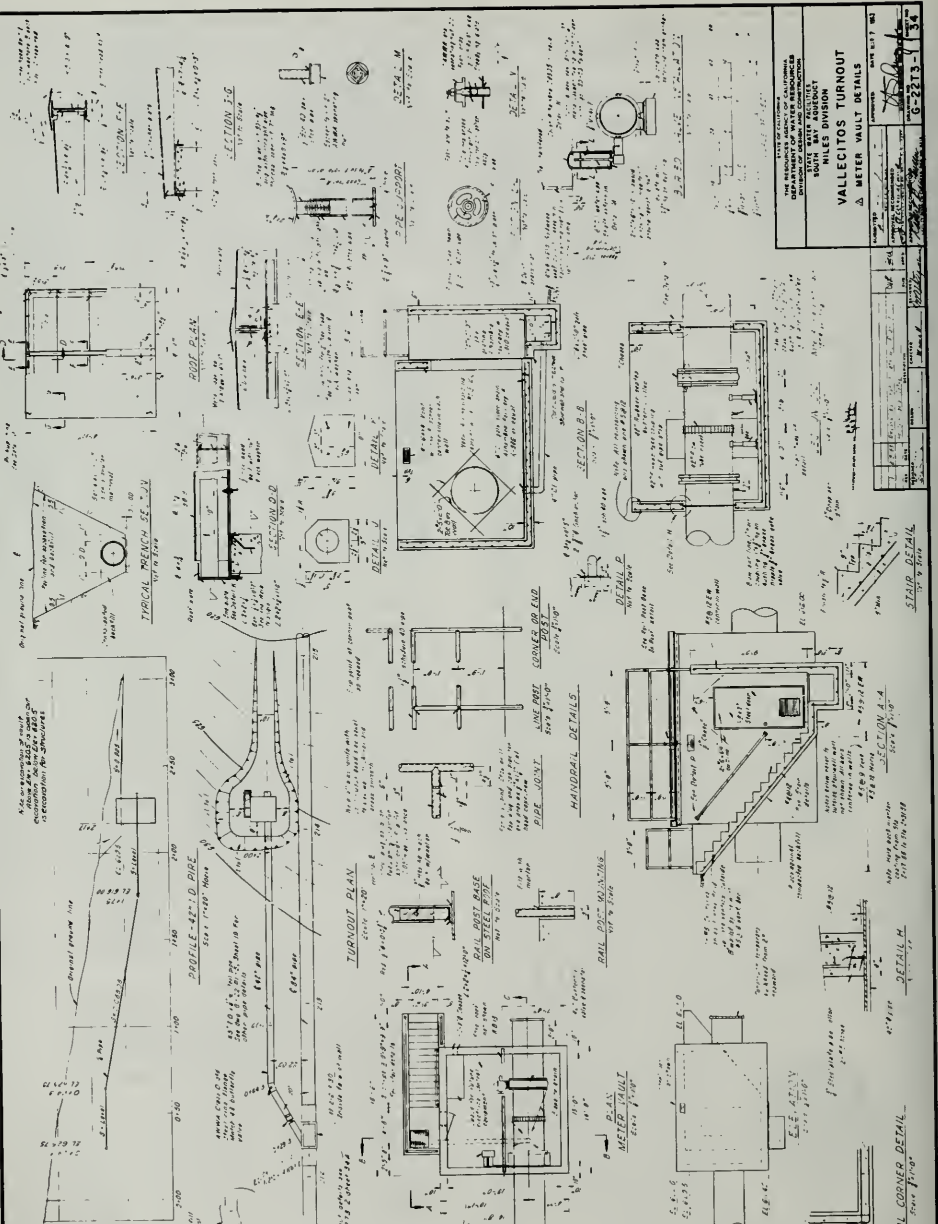

Figure 71. Vollecitos Turnout Meter House

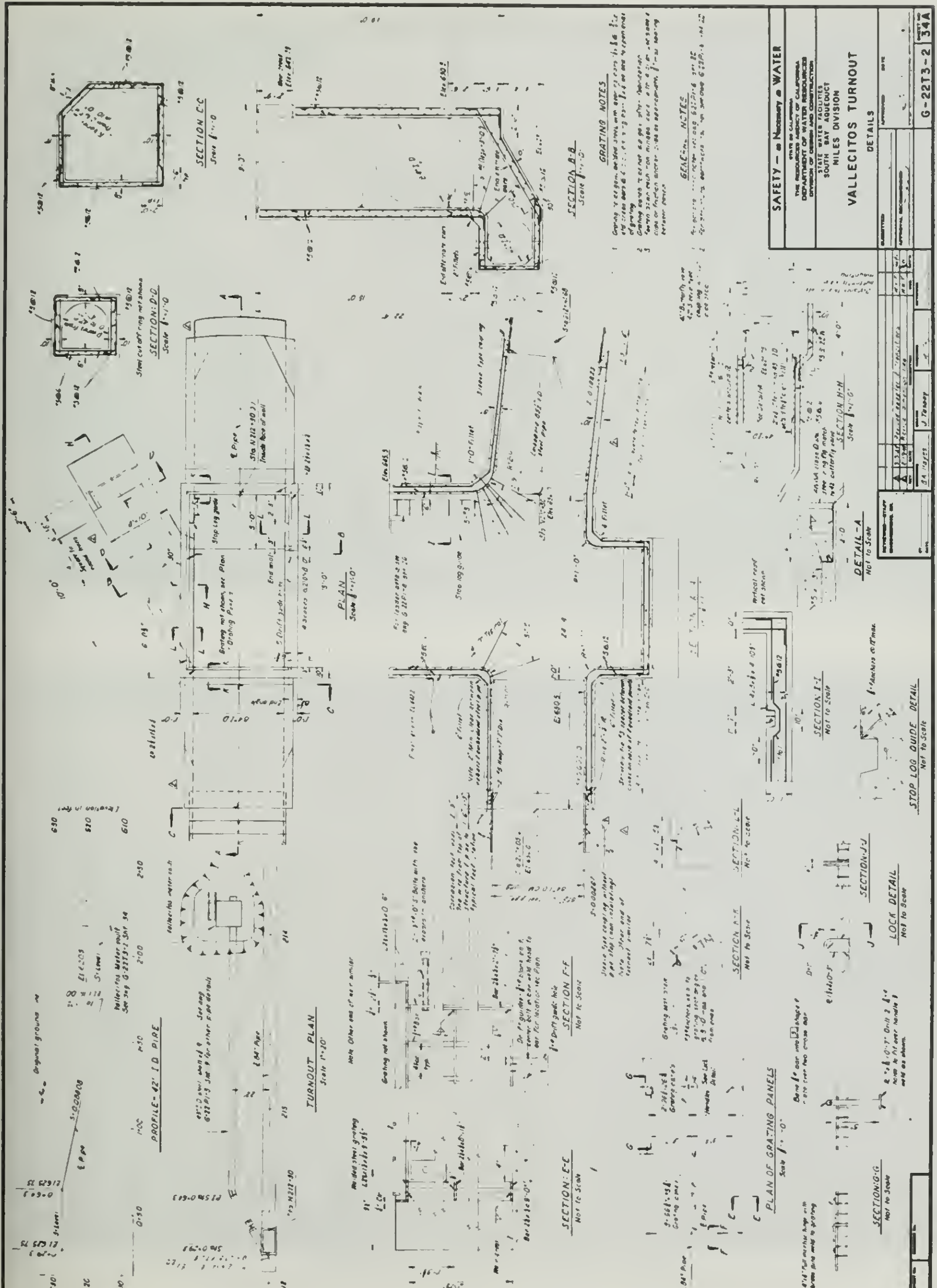

Figure 72. Vallecitos Turnout Structure

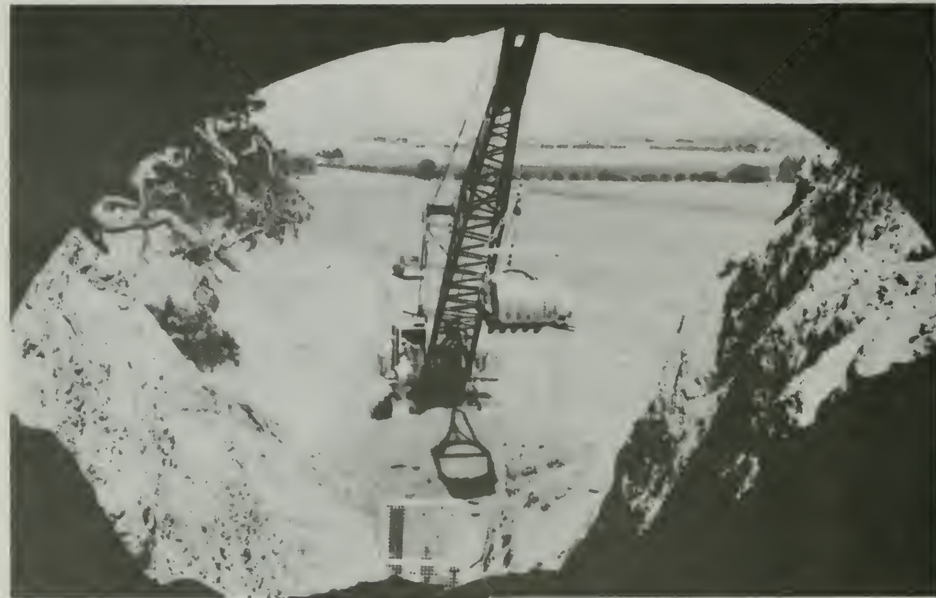

Figure 73. Pipe Trench Excavation—Del Valle Pipeline

Figure 74. Placing Pipe in Trench—Del Valle Pipeline

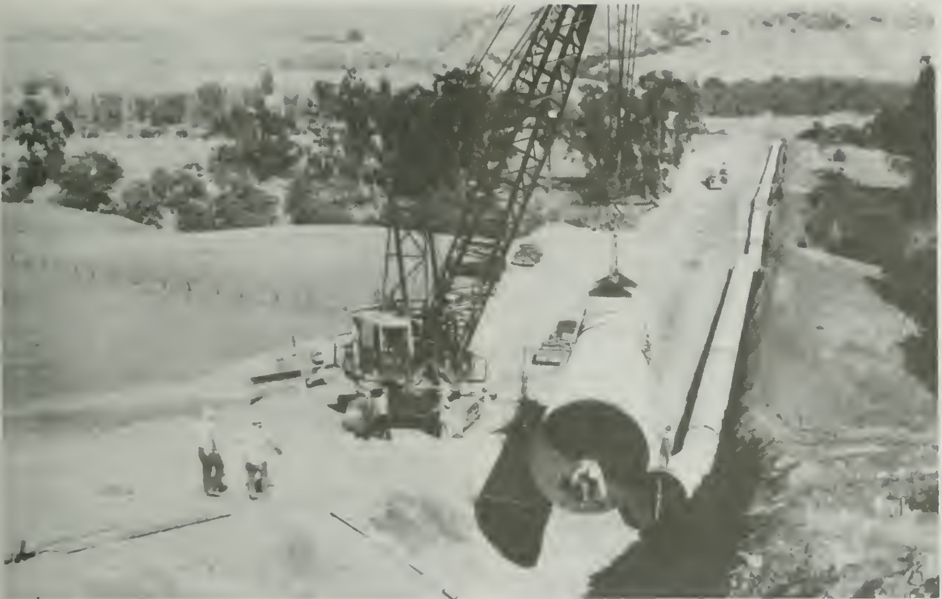

Figure 75. Lowering Pipe Into Trench—Del Valle Pipeline

Figure 76. "Stobbing" Top of Pipe for Tock Weld—Sunol Pipeline

Heavy rains in the winter of 1964 caused nearly 2,000 feet of pipeline, which had been backfilled with consolidated material, to float as much as 5 feet above grade (Figure 77). Water entering the trench contaminated the backfill with silt. After removing the backfill, a section of pipe was removed, and the remaining pipe gradually was lowered into place and strutted and jacked into shape.

To protect against further pipeline movement from high ground water, clay cutoff collars were placed at specified locations in the completed pipeline.

The grade of the free-flow section west of the south Livermore turnout upstream to La Costa Tunnel was of special concern. Because of the floated pipe that occurred in November, backfill also was removed to the springline of this section of pipe. The pipe was strutted, and any material loosened by this strutting was recompacted. Consolidated backfill then was placed to 1 foot over the top of the pipe, followed by whatever backfill was necessary. Above the pipe, backfill to the maximum cover of 10 feet was placed without difficulty.

Special closures were made in the steel pipeline with individually fabricated butt sections which, after welding, were coated with coal-tar epoxy. Connections between concrete and steel pipe at the Tunnels were made with special dresser-type couplings.

Mortar lining was applied in the field after backfilling. The pipe was first prepared with a rotary pipe cleaner, followed by a self-propelled lining machine. Mortar was applied with a centrifugally spinning "brush". Three spring-loaded trowels mounted on arms at the back of the machine rotated as the machine moved ahead and produced a smooth trowel finish (Figure 78). The lining machine was screw-fed from a concrete buggy attached behind the lining machine. Lining was water-cured with a self-propelled buggy

Figure 77. 84-Inch-Diameter Steel Pipe Floated by Rain Water—Sunol Pipeline

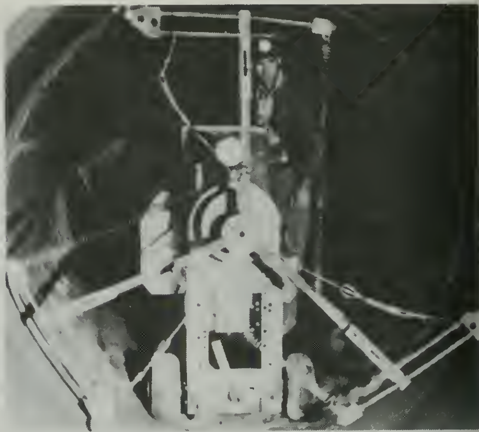

Figure 78. Pipe Lining Machine Applying Cement-Mortar Lining—Sunol Pipeline

on rubber-tired wheels powered by two 12-volt batteries. The buggy contained a 25-gallon water tank which fed a spray nozzle mounted on front of the machine. The water spray was actuated through a hand-operated air pump. This method proved quite satisfactory; however, it did not have sufficient power to operate on steep grades, and curing was accomplished by introducing water at the top of the slope and allowing it to flow down the pipe.

Del Valle Branch Pipeline.

Design. The Branch Pipeline, in conjunction with Del Valle Pumping Plant, conveys water from the main-line South Bay Aqueduct during periods of low water demand to Lake Del Valle for storage and conveys stored water from Lake Del Valle to the Aqueduct during periods of peak water demand. This transfer can be accomplished in either direction by pumping or by gravity flow. The direction of gravity flow depends on the relative water surface elevation in the Lake and at the South Bay Aqueduct diversion to Del Valle Branch Pipeline.

The Del Valle branch system was designed to have the capability to (1) convey 120 cfs out of Del Valle reservoir until the reservoir water surface reaches a minimum elevation of 639 feet, and (2) conveys 120 cfs into the reservoir until the water surface reaches a maximum elevation of 704 feet (Figure 79).

The Branch Pipeline is a 60-inch-diameter, prestressed-concrete, buried pipe that extends from the Del Valle Branch wye on the main-line South Bay Aqueduct at Mile 18.6 to Del Valle Pumping Plant, a distance of 1½ miles. Pipeline appurtenances are a main-line, shutoff (nonregulating), butterfly valve and vault; a 6-inch turnout with valves and a flowmeter; one 30-inch turnout pipeline (with valves, flowmeter, and dissipator); one surge tank equipped with a shutoff valve (nonregulating); two blowoffs; three air valves; one pitot tube vault; nine corrosion test stations along Del Valle Branch Pipeline; and a cathodic protection system at the juncture of the Branch Pipeline and Del Valle Pipeline.

The 60-inch pipe was designed for alternative bids for concrete cylinder pipe; prestressed-concrete cylinder pipes; or mortar-lined and coal-tar-coated, welded, steel pipe. The design head ranges from 150 feet to 250 feet with the design earth cover ranging from 7 to 15 feet. The Pipeline and appurtenances were designed according to criteria described earlier in this chapter.

The tank is located on a hill about two-thirds of the way between the branch turnout on the main-line aqueduct and Lake Del Valle. The tank is connected to the Branch Pipeline by a short length of 60-inch pipe that contains a 60-inch, shutoff, butterfly valve (Figure 80).

The tank, which is 17 feet in diameter and 60 feet in height, was fabricated from ASTM 283C Steel Plate, ASTM A36 Structural Shapes, and ASTM A354 B. C. anchor bolts. The tank, was designed according to AWWA Standard for Welded Steel Elevated Tanks, Standpipes and Reservoirs for Water Storage—D100-65 using a 100-mph lateral wind load and a 0.2g lateral seismic load. The 60-inch butterfly valve was specified to meet the requirements of AWWA Standard C504.

The tank was sized to (1) supply an adequate storage to contain critical upsurges in the event of power failure when pumping water into the reservoir at flow rates in excess of 120 cfs, and (2) contain a sufficient volume of water to prevent water column separation in the event of power failure when pumping water from the reservoir at flow rates in excess of 120 cfs.

For gravity flows into Lake Del Valle and from Lake Del Valle (when reservoir water surface is below elevation 695 feet), the surge tank serves as a vent.

The 60-inch butterfly valve is located between the steel pipe tee on the Branch Pipeline and the surge tank to isolate the surge tank in case of gravity flow from Lake Del Valle when the reservoir water surface is above elevation 695 feet. In this latter case, vacuum relief and air evacuation is provided by a combination air release-vacuum valve atop the Branch Pipeline adjacent to the surge tank.

The 30-inch-diameter, turnout, coal-tar-lined and coated, steel pipeline extends from a side outlet in the Branch Pipeline at the Pumping Plant some 445 feet to a dissipator located at the edge of the Del Valle Dam outlet channel. It is equipped with a 30-inch, shutoff, butterfly valve and a 30-inch flow tube and meter and terminates at a 24-inch hollow-cone valve located in the dissipator structure (Figures 81 and 82).

The turnout pipeline releases flows (120 cfs maximum) into Arroyo Del Valle from South Bay Aqueduct to satisfy a portion of the downstream water rights. In exchange, an equivalent amount of Arroyo Del Valle streamflow is stored in Lake Del Valle at a higher energy level. This exchange reduces the cost of pumping this water into Lake Del Valle.

Construction. This pipeline connects Del Valle Pumping Plant with Del Valle Pipeline.

The trench was excavated with a crawler backhoe. Ground water was encountered throughout a major portion of the trench which required overexcavation and replacement with bedding material.

Prestressed-concrete cylinder pipe was used and placed with a crane (Figure 83). After installation, the rubber-gasketed joint was mortar-lined and grout-filled. Consolidated backfill material was placed to a height of 18 inches above the pipe invert.

Del Valle Branch Pipeline across Arroyo Del Valle was encased in concrete for added stability (Figure 84).

The surge tank was fabricated and erected in eight rings of three segments each (Figure 85). The tank was field-welded, radiographed, painted, and successfully pressure-tested.

Sunol Blowoff.

Design. This facility is located in Sunol Valley upstream of the Hayward fault and will be used to waste in-transit aqueduct flow should an emergency condition occur in the Santa Clara Pipeline, or in certain cases where the rejection of scheduled upstream turnout flows would create flows in excess of the aqueduct capacity beyond Mission Tunnel.

This blowoff was sized to waste the full pipeline capacity beyond Mission Tunnel. The blowoff can be activated automatically by a pressure sensing device to ensure that the downstream aqueduct capacity will not be exceeded and also to prevent overtopping of the standpipes along Santa Clara Pipeline. It can be operated remotely to provide a quick shutdown of flow to Santa Clara Pipeline.

Sunol blowoff (Figure 86) extends from a standard, flanged, manhole nozzle in Sunol Pipeline and includes a flanged 20-inch rotovalve connected to a section of 20-inch steel piping. The steel piping is connected to a horizontal, 20-inch, hollow-cone valve which is mounted in the wall of a dissipator box. Flows through the hollow-cone valve discharge into a 12-foot by 17-foot by 12-foot-deep, concrete, dissipator box. The dissipator box is provided with a wingwall outlet measuring 9.5 feet by 11.5 feet in height which discharges the flow into Alameda Creek channel. The wingwalls and apron form a cold joint with the dissipator box to accommodate differential settlement.

Construction. Because an earthquake could rupture aqueduct facilities in the western reaches, it was decided to install an emergency release in Sunol Pipeline just east of Mission Tunnel. The blowoff, as previously discussed, was constructed without incident (Figure 86).

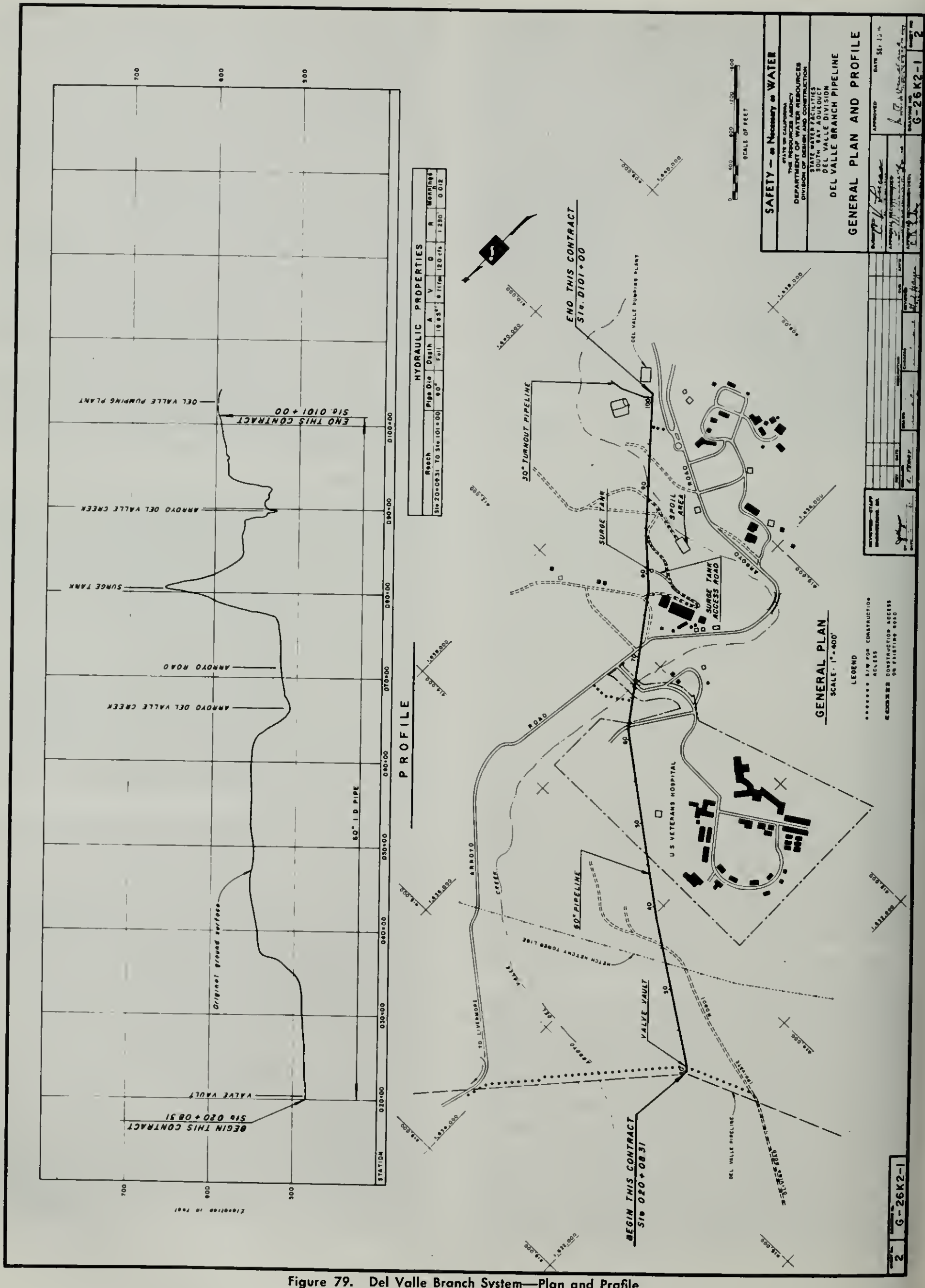

Figure 79. Del Valle Branch System—Plan and Profile

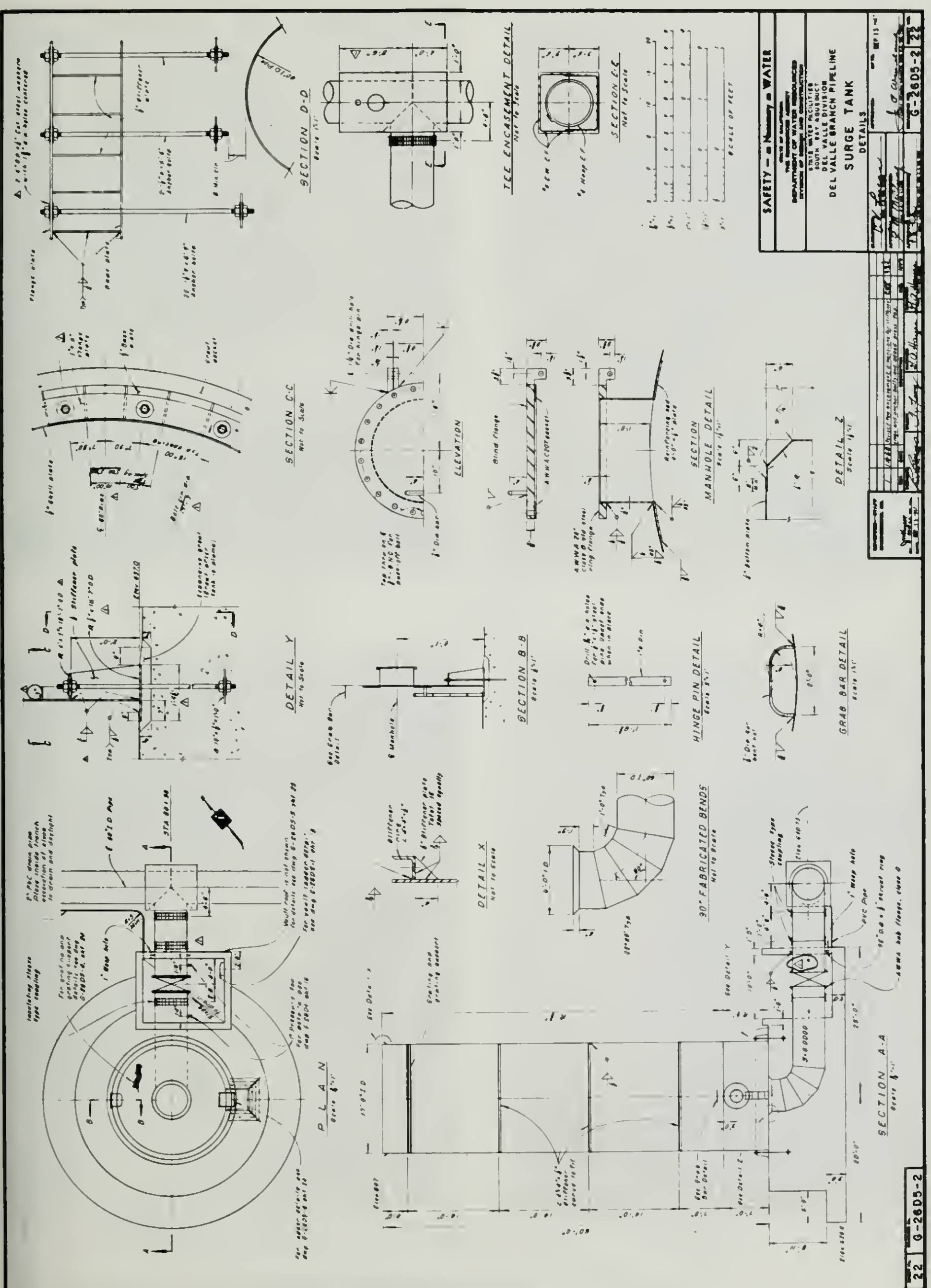

Figure 80. Surge Tank—Del Valle Branch Pipeline

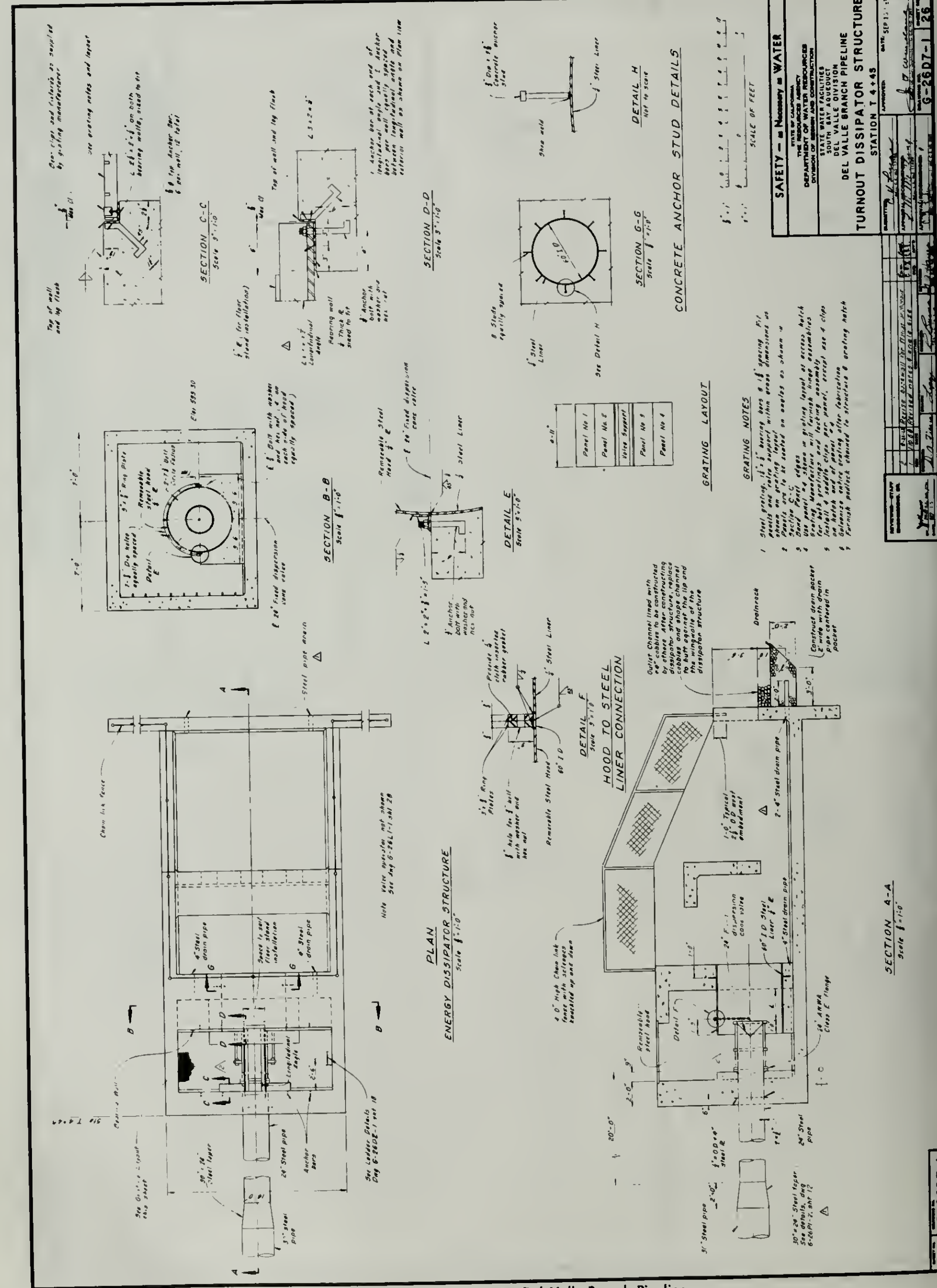

Figure 81. Dissipator Structure—Del Valle Branch Pipeline

Figure 82. 30-Inch Turnout Dissipatar in Operation—Del Valle Branch Pipeline

Figure 83. Crane Lawering Pipe Inta Trench—Del Valle Branch Pipeline

Figure 84. Concrete Encasements at Arroyo Del Valle Creek—Del Valle Branch Pipeline

Figure 85. Surge Tank Erection—-Del Valle Branch Pipeline

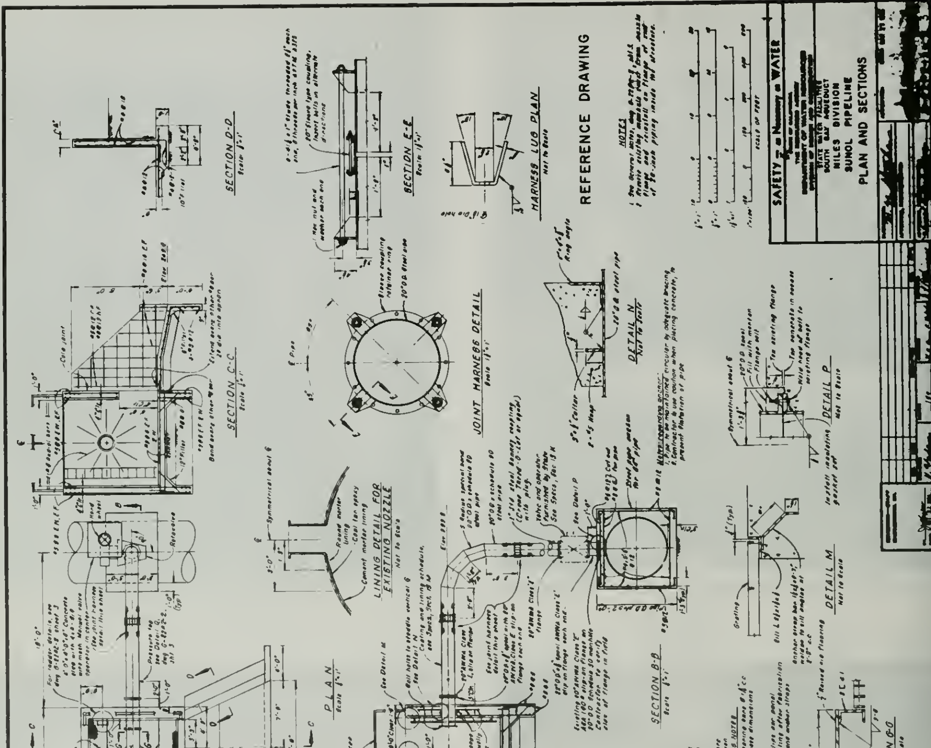

Figure 86. Sunol Emergency Blowoff

La Costa and Mission Tunnels .

Design. Both tunnels are vented at the inlet and outlet portals and were designed for open channel flow at a depth of 0.82 of the tunnel diameter. La Costa Tunnel is 1 mile long, has a capacity of 305 cfs, and has a slope of 0.006. Mission Tunnel is 0.7 of a mile long, has a capacity of 255 cfs, and has a slope of 0.00112. Both tunnels were designed as a horseshoe section with the tunnel lining either a 7-foot - 6-inch horseshoe or a 7-foot - 9-inch circular section, if cast in place, or a 7-foot - 6-inch-diameter for precast sections (Figure 87). Flow requirements could have been met with a smaller tunnel section without appreciable head loss; however, because of limited working space, it would have been more expensive.

Geologic exploration indicated variable tunneling conditions, including possible squeezing conditions in La Costa Tunnel and blocky faulted conditions in Mission Tunnel. The rock support system specified was 6- by 6-inch, M20, steel, horseshoe sets spaced on 4-foot centers.

The horseshoe-shaped section was based on an 8-inch unreinforced-concrete lining. No reinforcement was considered necessary because the lining would be in compression. Steel tunnel supports were to be embedded in the lining, but no allowance was made for the additional strength they would provide. No tensile reinforcement was considered necessary at the corners because of the increased concrete section and the large, axial, compressive force on the section. Crushing or shear at the corners was considered resisted by the depth of the concrete section.

If a circular cast-in-place lining was used, the diameter was increased 3 inches to compensate for the reduced cross-sectional area assuming no reduction in the friction factor. For precast pipe, the reduced friction factor would offset the loss in the cross-sectional area and a 7-foot - 6-inch section was retained.

Cut-and-cover sections leading to and from the tunnel portals were specified as concrete pipe, and the portal sections were designed as cast-in-place reinforced concrete. Portal sections were defined as the transitions from open-cut trench where the loads had little or no side restraint, whereas in the tunnel, the loads had full side restraint. The design of these sections was based on the column analogy method of analysis for unsymmetrical sections. The design of the precast sections was based on the ultimate strength theory.

Backfill grouting of the voids between the lining and the excavated surface was specified throughout the length of the Tunnels. Grout connections were provided at 8-foot centers alternating 15 degrees right and left of the tunnel centerline. Grouting pressures were limited to 50 psi. Additional grouting connections were specified where overbreak was high.

Nine bids were submitted, all for the cast-in-place tunnel lining alternative. The contractor used the 7-foot - 9-inch-diameter, cast-in-place, circular, lining section.

La Costa Tunnel .

Construction. This tunnel was driven from the west portal. The portal cut was made on slopes of 1½:1 without benches since the maximum cut was only 46 feet.

The Tunnel is located in the Livermore formation, which consists of clays interbedded with silty sandy and gravelly clays. Ground conditions varied from "slow raveling", to "running", to "squeezing" in a few places.

The ground was never hard, and 5% of the excavation was made with air spades or hand tools (Figure 88). Holes were drilled with individually hand-held, air-driven, soil augers 6 feet long. The number of holes varied from 30 to 50 per round and were loaded with a combination of powder and 45% blasting gelatin. Most rounds were shot tight and trimmed to size with air spades to minimize overbreak. However, overbreak averaged 11% for the entire tunnel.

Tunnel supports were carried close to the tunnel faces, (Figure 89) and extensive lagging and blocking was required, particularly where gravelly ground was encountered. Timber crown bars, when required, were carried ahead of the last set, and breastboarding was employed occasionally. Shear zones from a few inches to 30 feet wide were encountered, some of which probably were the result of local faulting. In one shear zone, eight intermediate or jump sets were required for ground control and, in two other shear zones, two additional sets were required. Otherwise, normal blocking and lagging provided the necessary support. Invert struts also were used.

Ground water was never a problem, and the maximum flow was 10 gallons a minute. What ground water was encountered usually came from isolated sandy lenses which drained rapidly. Muck was excavated by an air-operated loader and placed into cars which were dumped at a designated spoil area adjacent to the portal cut. Gas was never a problem. The Tunnel was holed through in 145 working days.

Prior to concreting, all sets were checked for looseness and specified tolerances. The lining was placed from the inlet portal in a downstream direction.

A concrete batch plant, with fully automatic scales and a 3-cubic-yard mixer, was set up outside the downstream portal. The lining equipment train consisted of a specially fabricated, 96-foot, movable form and a single-throat pumpcrete machine with an 8-inch-diameter slickline pipe between the jumbo form and the tunnel walls (Figure 90). The holding hopper of the pumpcrete machine was fed by conveyor belt from two concrete holding cars, which in turn were fed by concrete carrier cars shuttling between the batch plant and the lining train. The jumbo form was in five panel sections which, for movement, folded inward sufficiently to permit it to be winched to the next position. The upstream end rested on the previously placed lining.

A 96-foot section was completed every 24 hours in three shifts: one for cleanup, one for form moving, and one for concrete placement during the day shift. Form vibrators were attached to the jumbo at the invert, ribs, springline, and crown. Frequent and uniform form vibration close to the concrete flow line produced a smooth lining. Immersion vibrators, operated from access hatches in the form, also were used. Following any hand patching required, a curing compound was sprayed on the lining. Grout was injected with an air-powered reciprocating pump, through a manifold to which 2-inch grout lines were connected. Bulkheads placed at the tunnel openings decreased the flow of air and increased the humidity in the Tunnel for better concrete curing.

Mission Tunnel.

Construction. This tunnel was driven from the west portal after constructing an access road and excavating for the portal. To minimize slide potential, the exit portal excavation included two 20-foot benches at 30-foot vertical intervals. In spite of the benching, a slide occurred above the first bench. The slide, 50 by 45 feet and about 5 feet deep, moved slowly and after a few weeks stabilized. After ditching for drainage, the slide was left in place.

The Tunnel was excavated by drilling and blasting. A jumbo with two rock drills was used. The drill jumbo also was used for erection of the tunnel support system. Steel sets were similar to those used in La Costa Tunnel. Spacing varied from 2 to 7 feet, with an average of 5 feet, and invert struts were necessary to restrain the legs of the sets.

Nearly 500 feet of tunnel near the east portal was driven without support. However, occasionally, the formation was closely fractured, friable, very blocky, and seamy rock. In local faults, the rock was brecciated, crushed, friable sandstone with many slickened clay seams which, on occasion, resulted in squeezing ground.

The Briones formation was at times strongly jointed on a random pattern, but the most distinctive joint sets were at right angles to the tunnel bore. Some shears were 6 to 8 inches wide. Where the formation was extensively sheared, it also was water bearing. One flow of 800 gallons per minute initially occurred on breaking into an open-jointed area (Figure 91). After a few days, this flow gradually decreased. One flow remained at 50 to 100 gallons per minute during its total period of exposure.

Ground water entered the Tunnel in sufficient volume to lower the local ground water table, as demonstrated by two springs drying up and another reducing in flow by over 50%.

Prior to lining the Tunnel, the high ground water inflow was channeled into a 4-inch, gravel-bedded, perforated, drain line located below the tunnel invert. The persistent high flow was delivered by special piping into the Tunnel through a weep hole. The Tunnel was lined in similar fashion to La Costa Tunnel, using the same batch plant and movable jumbo liner form.

Grouting of Mission Tunnel was similar to La Costa Tunnel, except a special effort was made to seal off ground water inflow. Normal contact grouting helped but, when the ground water level rose, the increased hydraulic pressure created leaks through the construction joints in the lining. Additional grouting sealed these joints, as well as the drainage channel in the tunnel invert.

Six months after completion of the Tunnel, the ground water level had recovered halfway to the preconstruction level. Flow into the Tunnel, however, had increased and three springs had not returned to normal flow. Because this change in ground water level was adversely affecting property owners, a separate contract was awarded for additional grouting. Remedial work consisted of injecting grout curtains at the ends of the aquifers and fractured rock zones. The curtains were formed by drilling eight holes through the lining equally spaced around the circumference of the Tunnel to a depth of 10 feet. At points of known ground water inflow, additional grout was injected through four holes equally spaced around the circumference of the Tunnel. Grouting pressures were limited to 100 psi.

Cracks and construction joints were selectively grouted. Holes usually were slanted to intersect cracks at the end of the hole. Grouting was continued until the crack returned grout or the hole refused to accept grout.

The additional grouting successfully reduced ground water inflow into the Tunnel to 1 gallon per minute. Within six months of completion, the affected springs had returned to preconstruction flows, and the level of the ground water table nearly had recovered.

Mission Tunnel to Santa Clara Terminal Facilities

Santa Clara Pipeline extends from the exit portal of Mission Tunnel 12.8 miles to the South Bay Aqueduct terminal facilities.

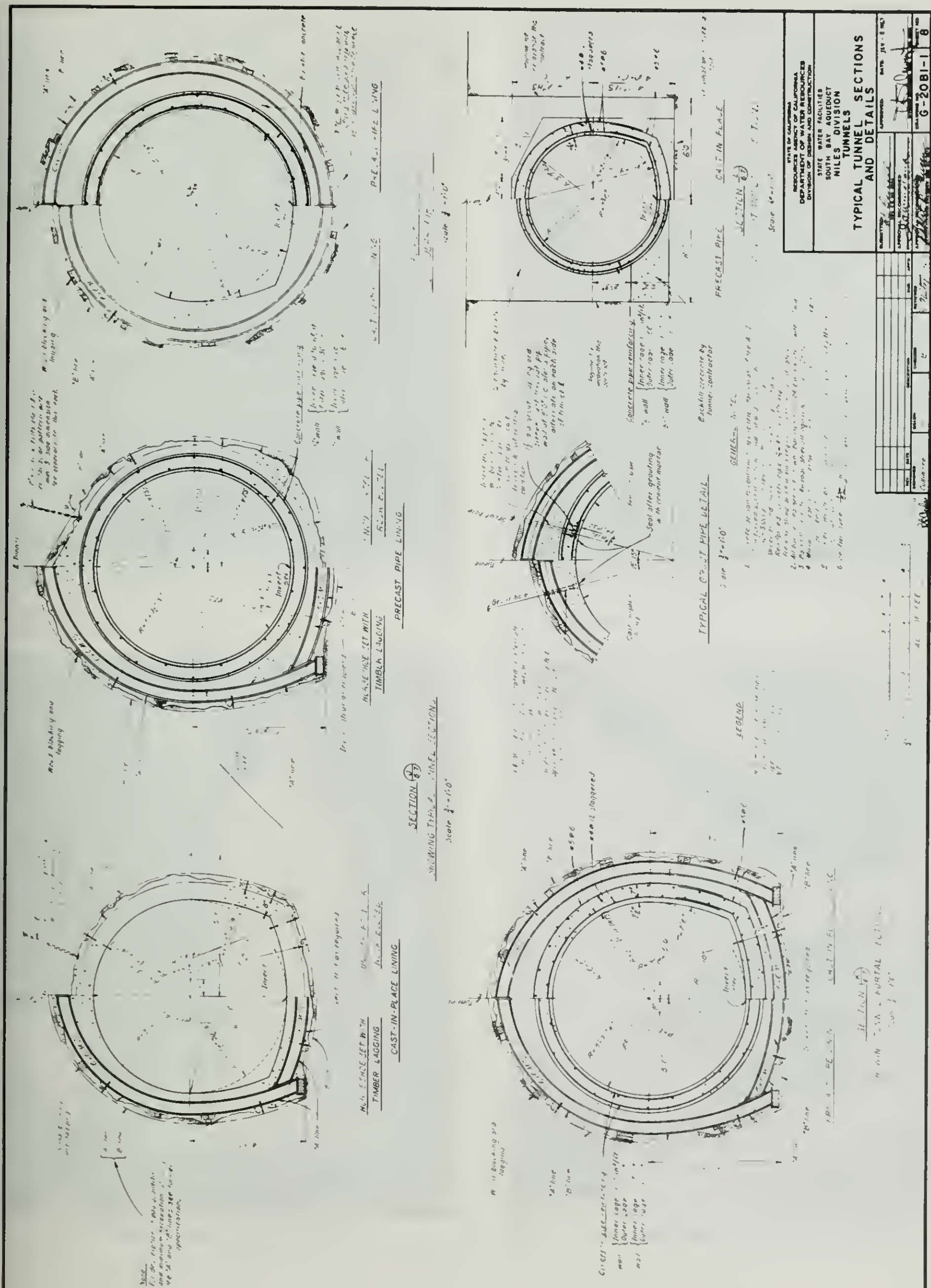

Figure 87. Typical Tunnel Cross Section—Mission and La Costa Tunnels


Figure 88. Excavated Gravel Face—La Costa Tunnel

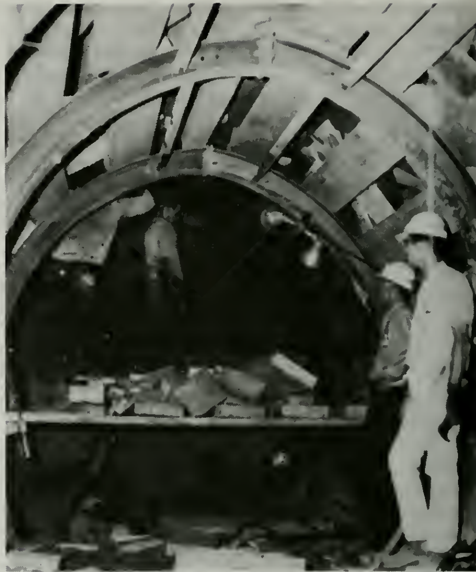
Figure 89. Installing Lagging and Blocking Behind Steel Sets—La Costa Tunnel

Figure 90. Slickline Behind the Tunnel Lining Form—La Costa Tunnel


Figure 91. Flow at Heading—Mission Tunnel

Santa Clara Pipeline .

Design. The Pipeline for the first 10 miles traverses the low ridges and draws of the foothills above Santa Clara Valley. The final 2.8 miles are located on the valley floor and pass through orchards and residential areas (Figure 92).

Along the higher portion of the alignment, sandstones, siltstones, shales, and silicious shales of the Oakland, Monterey, and Briones formations were encountered. Along the lower portions of the alignment of the Santa Clara formation, clays and alluvium were encountered. The Pipeline closely parallels the Hayward fault for approximately 9 miles and, at several places, crosses fault traces that probably are within the fault system. The Hayward fault is active, and creep has been reported as close as a few miles to the north.

The Pipeline was designed for alternative bidding on steel or concrete pipe. Design criteria and specifications were similar to those for the Del Valle and Sunol Pipeline reaches, except that the steel pipe joints would be rubber-gasketed-bell and Carnegie-spigot-ring. In cases of exceptional alignment or grade changes, welded joints were required. Maximum grade varies from a downslope of 53% to an upslope of 72%. In cases where backfill exceeded 10 feet, steel pipe was to be vertically braced on 10-foot centers before placing the backfill. Of nine bids received, three including the low bid were for steel pipe. Field-applied mortar lining was used.

One turnout stub, the Alameda Bayside turnout, has a 16-cubic-foot-per-second design capacity and is located about 3,000 feet west of Mission Tunnel. The turnout stub, which is closed with a blind flange, consists of a 20-inch nozzle extending from the Pipeline.

At Santa Clara County line, an in-line flowmeter was installed because all water deliveries beyond that point are made to the Santa Clara Valley Water District. The metering device consists of a 72-inch Dall flow tube and appurtenant metering equipment installed in a reinforced-concrete meter vault.

The Westside wye is a 45-degree branch to the Santa Clara Valley Water District's main pipeline. It was designed with a three-plate reinforcing system based on the AWWA publication, "Design of Wye Branches for Steel Pipe", June 1955. The wye is encased in a reinforced-concrete anchor block (Figure 93).

Construction. The prime contractor fabricated the steel pipe and subcontracted all other contract items.

Pipe trench was excavated in two zones: Zone I with side slopes of ¾:1, and Zone II with ½:1 slopes. Initially, scrapers were used but they were replaced by backhoes. On steeper grades, the backhoe was operated from a movable platform anchored with a dozer to maintain a reasonably level position (Figure 94). In accordance with a local ordinance, 2:1 cut slopes were used within the city limits of Fremont.

Pipe sections were fabricated at a plant in Napa from sheet steel produced at a steel mill in Fontana. The basic segment was an electric-welded 20-foot section which was joined in 40-, 60-, or 80-foot lengths for field installation. Most of the sections were 80-foot lengths weighing approximately 32 tons. About 80% of the pipe joints were Carnegie-ring type with rubber gasket. All nozzles and manholes were shop-welded onto the pipe during fabrication.

Pipe closely followed standard procedures for rubber gasket, bell, and spigot concrete pipe. In joining the spigot to the previously laid bell, the ends were cleaned and lubricated and the lubricated gasket placed on the spigot end. Pipe was lowered into the trench by a crane (Figure 95); jacks, come-alongs, and pry bars were used in making the final fit. If the gasket was damaged during fitting, the joint was marked for welding to avoid delay in laying the pipe.

On steep grades, a track-supported saddle was used, operated by a crane upslope of the section being placed (Figures 96, 97, and 98). Pipe sections on steep grades were joined by electric arc welding. All butt-strap joints were welded both inside and out and either radiographed or checked with a dye penetrant.

About 1,200 feet of pipe trench in the vicinity of Toroges Creek was unstable because of rising ground water, and the trench slopes were flattened from ¾:1 to 2:1. A 24-inch-deep by 20-inch-wide subtrench was excavated in the bottom of the main trench, and an 8-inch, perforated, asbestos cement pipe was placed in this subtrench which was backfilled with select filter material. The top 3 feet of backfill was compacted to prevent entry of surface runoff.

Prior to backfilling all trenches, joints were field-wrapped, and any tears or breaks in the shop-installed exterior coating were repaired. Consolidated backfill was placed in two lifts: the first to the pipe springline, and the second to the top of the pipe. Both lifts were jetted and vibrated simultaneously. Loose or compacted backfill, as required, then was placed to the specified depth.

In December 1964 and January 1965, heavy and prolonged rains halted construction. Almost 3,000 feet of pipe floated as much as 8 feet and had to be relaid. This problem occurred in the vicinity of Agua Caliente and Agua Fria Creeks.

Lining was applied in similar fashion to that for the Del Valle and Sunol Pipelines. However, the mortar was forced into a rapidly revolving dispenser head and centrifugally applied rather than pneumatically. This method eliminated rebound and air-entrainment characteristics of pneumatic application and produced a dense homogeneous lining.

As soon as a section of pipe was completed, it was tested against an allowable loss during 24 hours of 100 gallons of water per mile of pipe. The pipe was filled for 15 days prior to the test to saturate the lining. The average loss was 42.7 gallons per day per mile of pipe.

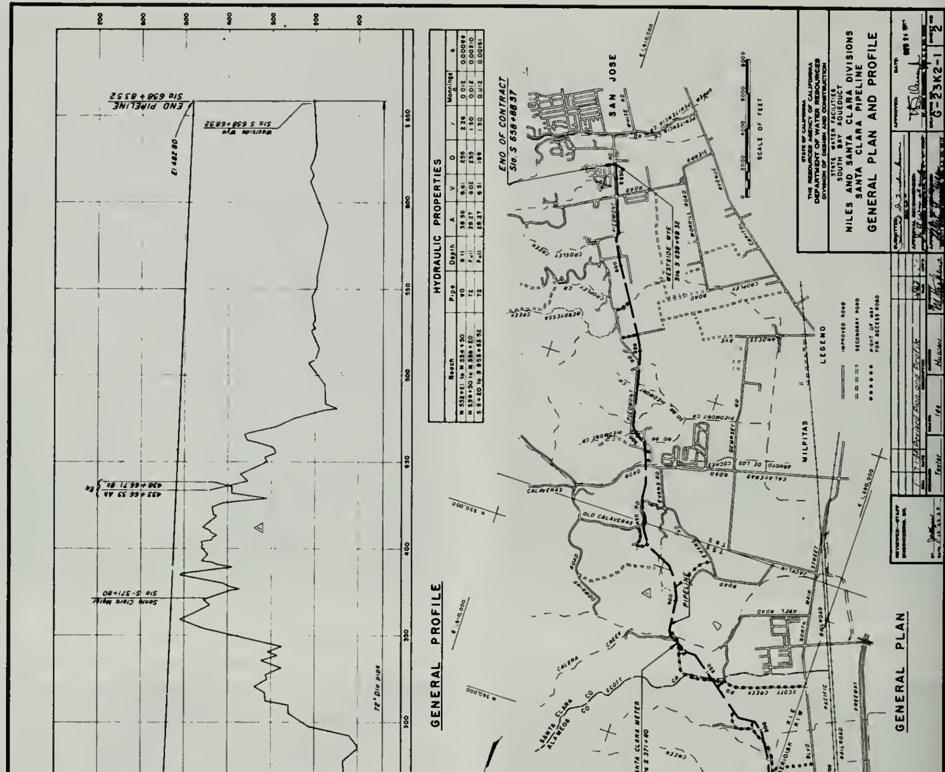

Figure 92. Santa Clara Pipeline—Plan and Profile

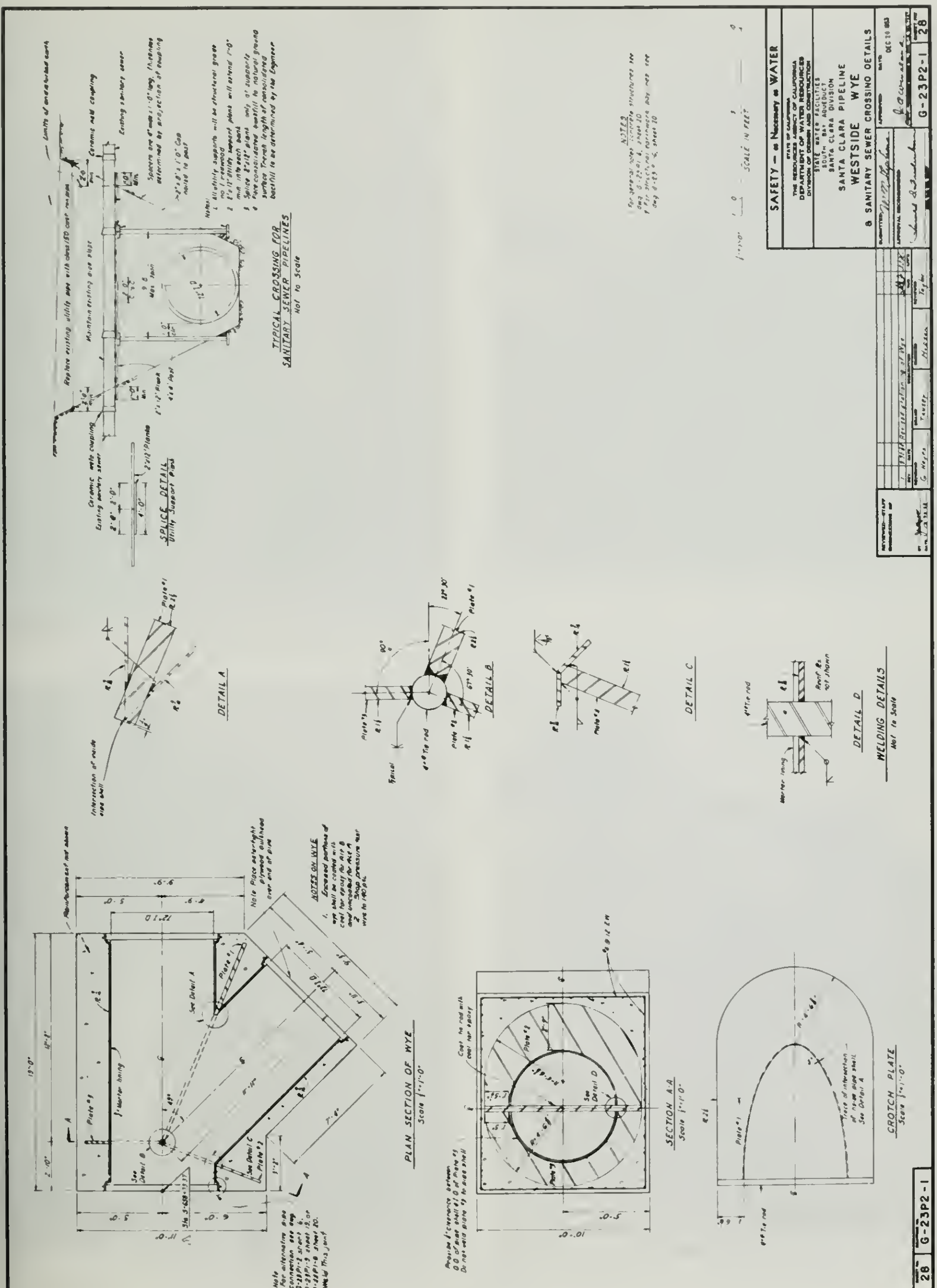

Figure 93. West Side Wye Details—Santa Clara Pipeline

Figure 94. Backhoe Operated From Movable Platform—Santa Clara Pipeline

Figure 95. Lowering 72-Inch Steel Pipe Into Trench—Santa Clara Pipeline

Figure 96. Steel Pipe Nested in Saddle for Installation on Steep Slope—Santa Clara Pipeline

Figure 97. Saddle With Steel Pipe Being Lowered Onto Railroad Tracks—Santa Clara Pipeline

Figure 98. Saddle With Pipe Guided Down Steep Slope on Rails—Santa Clara Pipeline

Santa Clara Terminal Facilities.

Design. South Bay Aqueduct terminal facilities are located in east San Jose on the site of the Santa Clara Valley Water District's Penitencia Treatment Facilities, which are served by South Bay Aqueduct. Two of the terminal facilities pipelines were constructed directly for the District's use. All of the terminal facilities were designed by the Department (Figure 99).

The facilities, starting at the end of Santa Clara Pipeline, consist of three parallel prestressed-concrete pipelines of 72-, 66-, and 60-inch inside diameters. The 72-inch pipeline connects to the Westside wye on Santa Clara Pipeline. These lines were spaced to allow placement in a common trench for approximately one-half mile to an overflow bypass structure. At the bypass, the 72-inch line continues for about 1,900 feet to the terminal, steel, storage tank. An overflow pipeline extending from the tank terminates approximately 3,400 feet to the south near Penitencia Creek at a percolation pond for a ground water recharge area.

The 66-inch line is a standby feed line for the future treatment plant, and the 60-inch line is a distribution line for treated water from the District's Treatment Plant.

The overflow bypass line directly connects the 72-inch inflow line and the overflow line, providing a bypass of the terminal tank (Figure 100). This allows emergency spillage into the overflow line and/or delivery of water to the District's percolation ponds without passing through the terminal tank.

Concrete pipe was specified rather than steel pipe since the more rigid walls of the concrete pipe do not require individual side support when the three parallel pipes are located in a common trench. Side support will not be required for future expansions.

Surface and immediate subsurface geologic conditions at the terminal facilities are of the Plio-Pleistacene Santa Clara formation. Santa Clara formation is mostly fat clay, some sand-silt mixtures, and some boulders from adjacent formations, usually in jumbled mixtures. The formation frequently is unstable and contains numerous landslides. The reservoir tank is located at the bottom slope of a gently rising hillside. An active slump area existed just upslope from the tank site prior to construction. Ground water was present within 25 feet of the surface over much of the area. A steel tank was selected over a compacted earth reservoir since no local deposits of embankment material were available in sufficient quantities.

The tank foundation consists of a concrete curbing 162 feet in diameter, enclosing 6 inches of selected topsoil on 18 inches of compacted backfill, placed over native material. The curb ring is continuously reinforced circumferentially and acts as a confining ring for the enclosed soil. Maximum design bearing pressure on the soil is 1,200 pounds per square foot. The tank has a floor-plate thickness of $\frac{1}{4}$ of an inch and 20-foot-high walls in three courses from $\frac{5}{8}$ of an inch thick at the bottom to $\frac{5}{16}$ of an inch thick at the top. Horizontal rigidity is provided by a steel-plate wind girder welded around the inside of the tank, 6 inches from the top (Figure 101).

To accommodate differential settlement, all piping was joined to the tank with flexible couplings. The inlet and outlet risers are $\frac{5}{8}$-inch steel-plate cylinders 132 inches in diameter by 17 feet in height. Risers were reinforced with circumferential stiffener angles at the top, bottom, and mid-height. A 1-inch neoprene pad between the bottom of the risers and the tank base plate prevents transfer of any vibrations from the risers to the tank walls. Height of the risers provides a column separation and permits the pipeline to be emptied without affecting storage in the tank (Figure 102).

An overflow line is controlled by an 18-inch butterfly valve near the base of the outlet riser which is hand-operated with a riser stem from a platform at the top of the riser. A 54-inch butterfly valve at the tank base is provided for future use. The tank can be drained through an 8-inch pipe controlled by a hand-operated butterfly valve into the 42-inch overflow line.

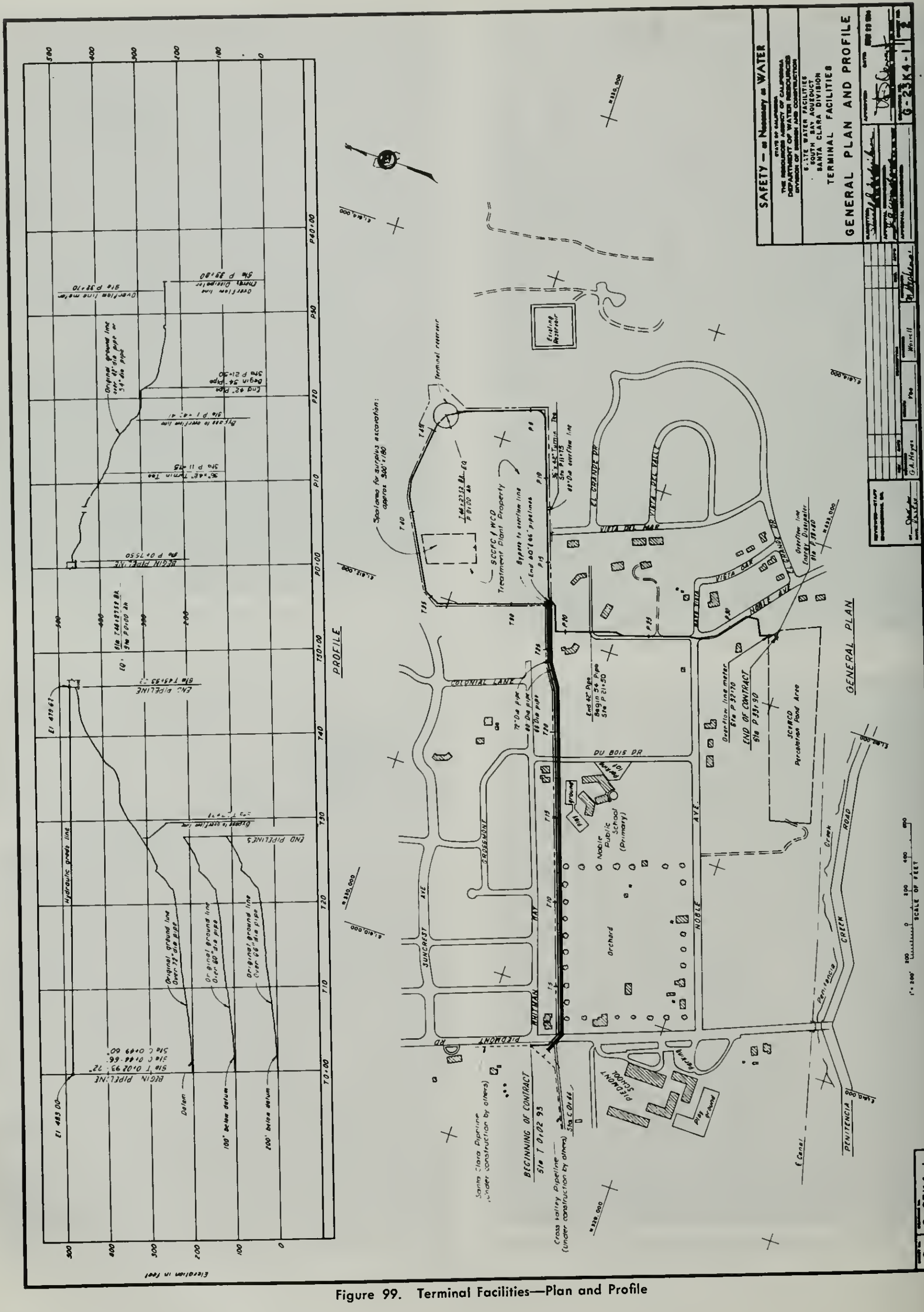

Figure 99. Terminal Facilities—Plan and Profile

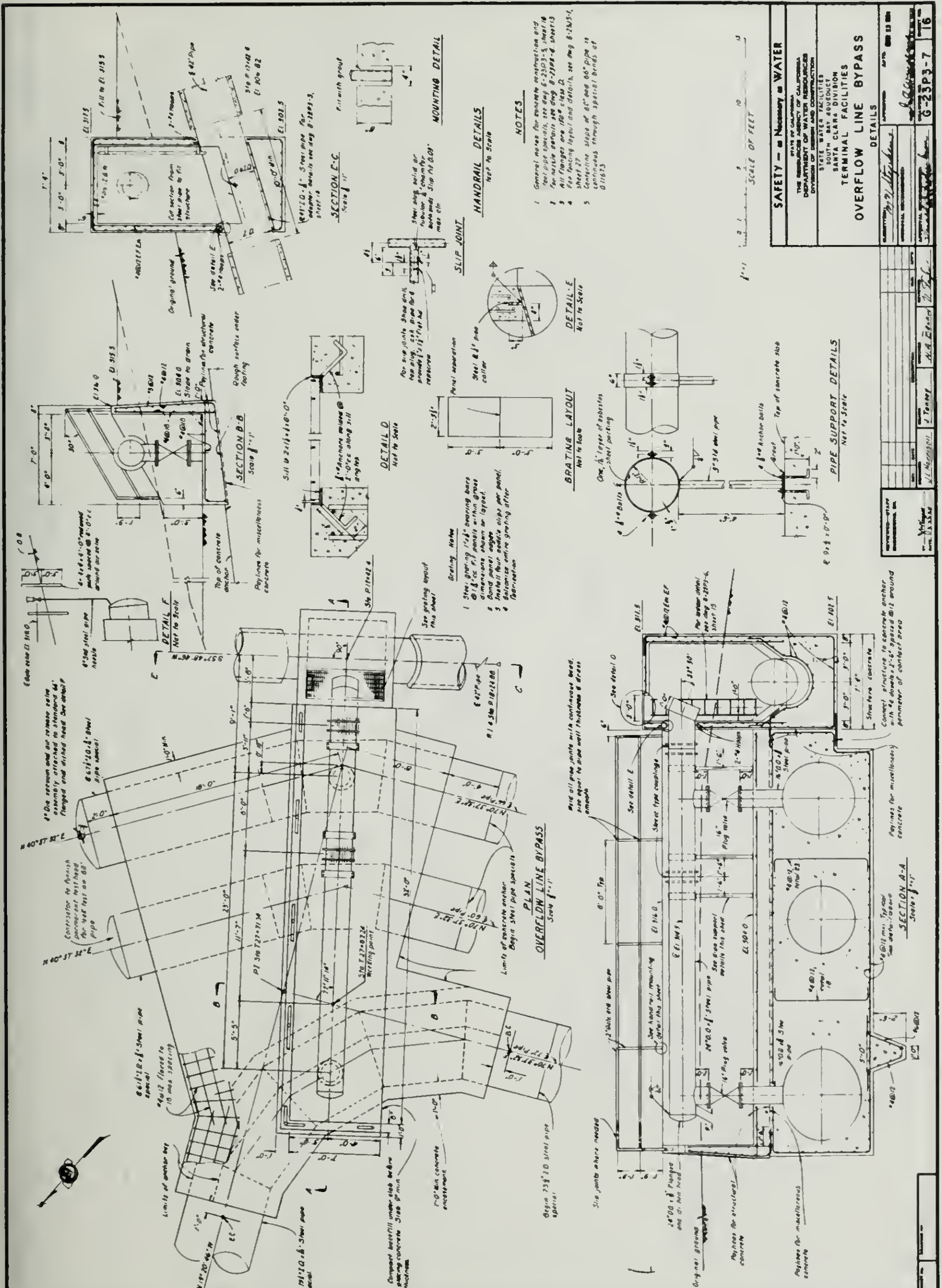

Figure 100. Terminal Facilities Overflow Bypass

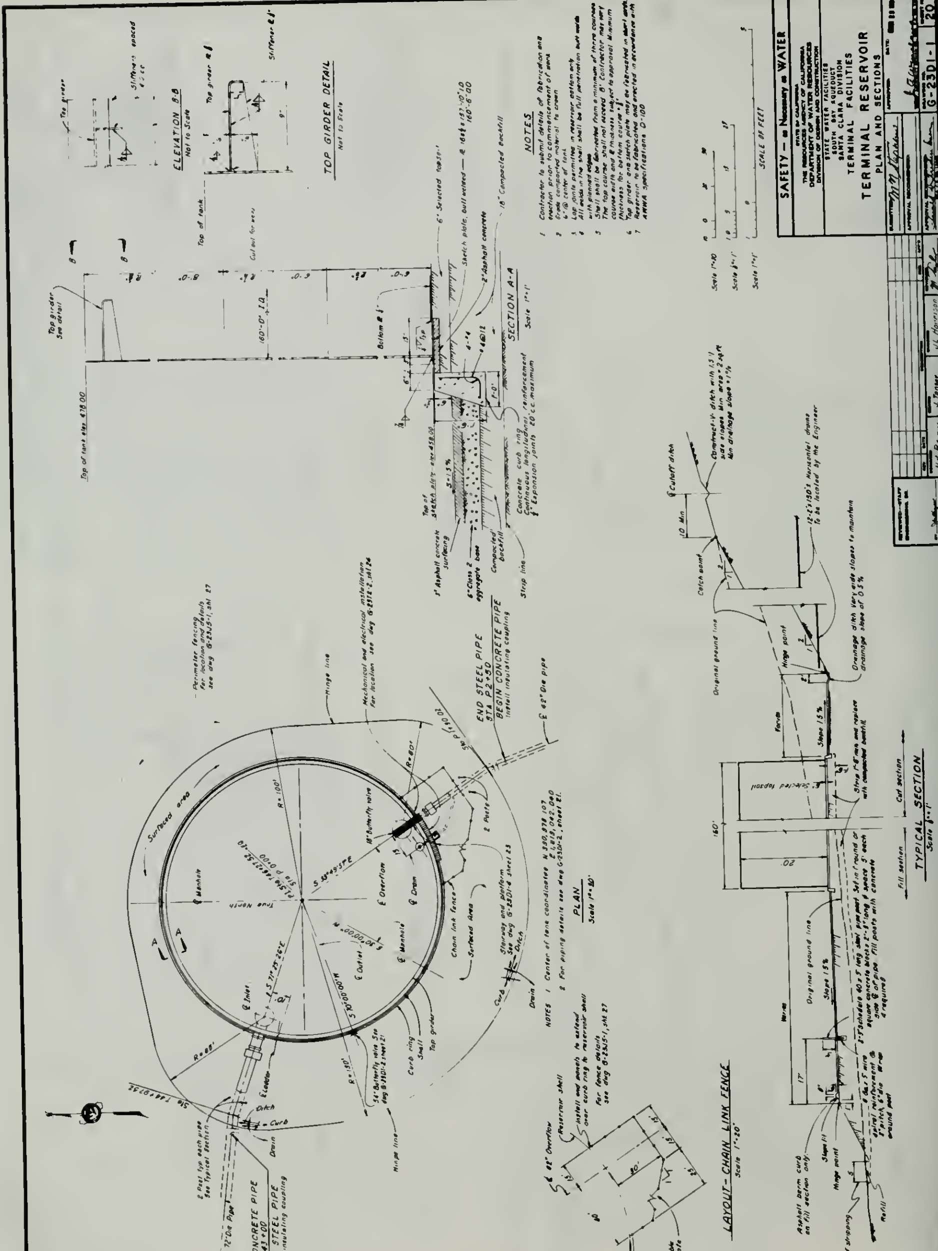

Figure 101. Terminal Reservoir—Plan and Sections

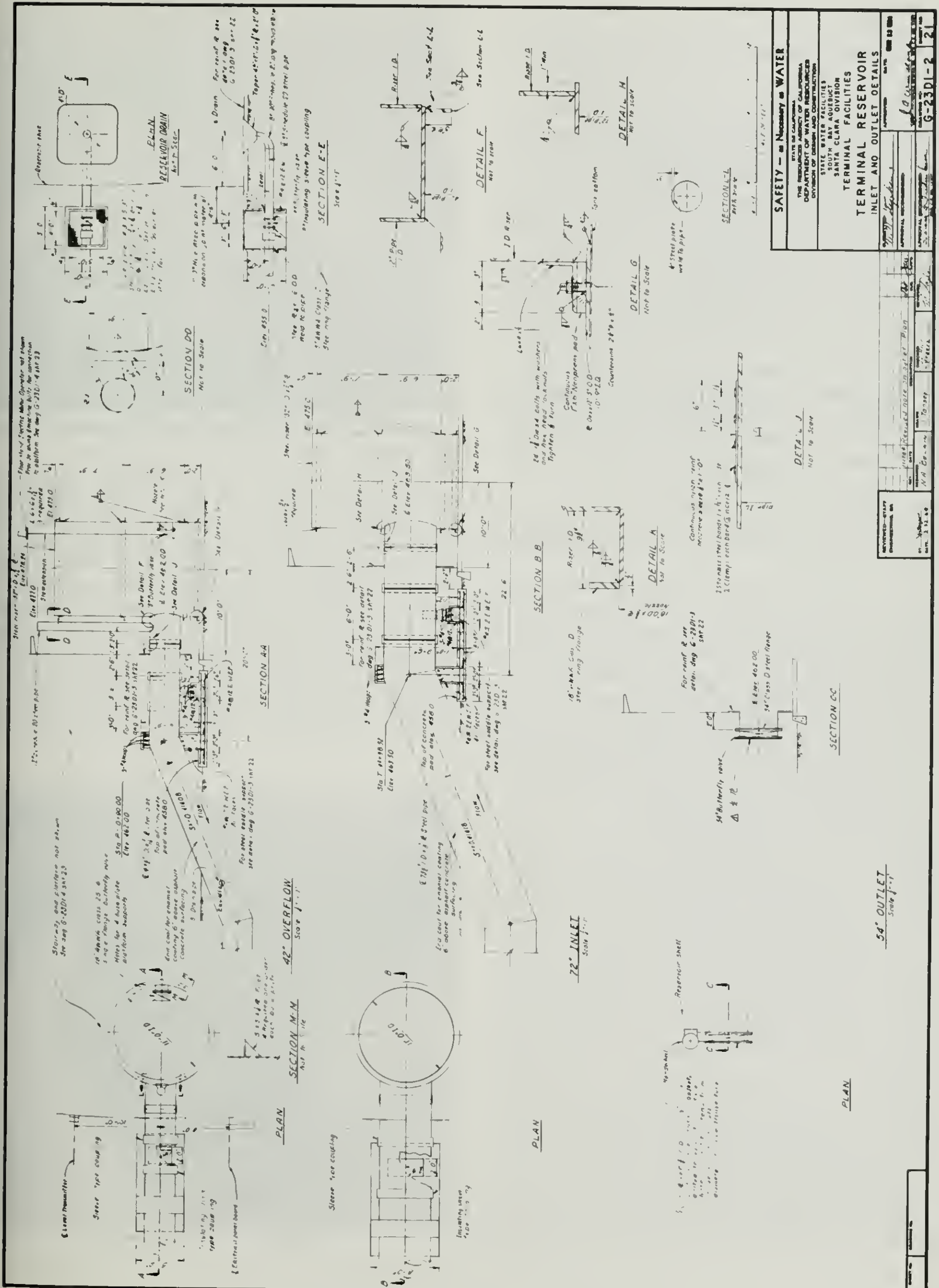

Figure 102. Terminal Reservoir Inlet and Outlet Riser Details

The overflow line extends from the terminal reservoir to the Santa Clara Valley Water Conservation District's percolation ponds on Penitencia Creek, a distance of approximately 0.63 of a mile. The pipeline consists of approximately 2,035 feet of 42-inch-diameter reinforced-concrete pipe and 1,230 feet of 54-inch-diameter reinforced-concrete pipe. Mortar-lined and coated steel pipe was used to form sharp bends. The main features along the overflow pipeline include a tee branch to serve the Treatment Plant, the overflow bypass structure, a 54-inch flow tube and meter, standpipes, and an energy dissipator.

The overflow line was designed for a maximum capacity of 184 cfs. Reinforced-concrete pipe was selected since the major part of the pipeline flows partially full. Analysis indicated a hydraulic jump would form in the overflow line creating blowback and, consequently, a 54-inch-diameter pipe was provided in the lower portion of the reach beginning at Station P21+50, and an air relief standpipe was provided at Station P19+60. For additional safety against blowback, a 12-inch air breather vent was located on the overflow line adjacent to the overflow riser.

The overflow line energy dissipator consists of a 9-foot section of 72-inch-diameter reinforced-concrete pipe, placed vertically in the bottom of the percolation pond.

Corrosion control for the terminal facilities was provided by an insulating joint at the Westside wye between the 72-inch concrete pipe and the steel pipe of Santa Clara Pipeline. Insulating joints also were provided adjacent to the steel tank to isolate the tank bottom from the buried pipe. Cathodic protection of the tank bottom was provided by high-silicon anodes installed around the periphery of the tank and a 50-ampere 30-volt rectifier. The tank bottom, except for the outside ring, was left unpainted. Zinc electrodes were provided under the tank bottom to monitor the effectiveness of the cathodic protection.

Construction. Pipe trenches were excavated with a dragline or backhoe. Although the 72-, 66-, and 60-inch lines, where parallel, were designed for multiple installation in a common trench, the contractor excavated and backfilled for each pipe separately. An exception to this was a 400-foot section leading to the bypass structure where right-of-way restrictions dictated a common trench for the 66- and 60-inch lines. Optional consolidated backfill was used from the springline to the top of the pipe rather than compacted backfill.

The hillside slope behind the terminal tank started moving downward soon after the tank base was excavated. Initial indications of failure appeared in the north slope as a slight protuberance of the finished cut slope. Measurements indicated approximately 1 inch of horizontal movement. The surface expression of the failure was well-defined within a month, and it appeared the tank site might be threatened. Movement appeared to be a simple rotational slide limited in both depth and surface area. The possibility of a large slide passing beneath the tank was considered, since the immediate area had a history of sliding. Approximately 5,000 cubic yards of material upslope from the existing cut was removed to stabilize the soil mass. However, slope-indicator measurements indicated continued movement of the slope, although at a slower rate. Twelve hydrauger holes were drilled 150 feet into the cut bank at the level of the tank base. Drilling water escaping through fissures in the slope confirmed the fractured and loose condition of the slope. Additional surface drainage facilities were added between the perimeter of the tank and the surrounding curb.

A subcontractor erected the tank during a period of heavy rains, which caused some problems. During the welding of the bottom plates, moisture was drawn up from the foundation surface sufficiently to produce unsatisfactory lap welds, and it was necessary to raise the plates to avoid moisture while welding.

Final welds proved very satisfactory, which was demonstrated by their ability to resist unplanned stresses later imposed. The outside ring for the bottom plates, which rested on the special asphaltic curb and gutter, was painted with epoxy. This coating could only be applied after the floor plates were welded. During the painting and curing process, it was necessary to raise and suspend the floor. The resulting stresses in the adjacent floor plate welds were never determined but undoubtedly were quite severe. After erection of the tank and before filling, diurnal temperature changes buckled the plates up to 6 inches. Lap

Figure 103. Terminal Reservoir

Figure 104. Terminal Tank Inlet Riser in Operation

welds, though subjected to repeated flexure from this distortion, suffered no damage. Some of the buckled floor plates did not regain contact with the foundation until the tank was filled.

After erection of the tank (Figure 103), an elevation grid was established on the floor, the top of the shell, and the tops of the inlet and outlet risers. The diameter of the tank was measured at the inside top of the shell at three separate locations. A check of these measurements demonstrated the tank and surrounding concrete curb were slowly but uniformly rising. No distortion of the tank has occurred. These movements have not precluded use of the facilities; however, surveillance is being maintained for movement of both the tank and the hillside.

South Bay Aqueduct was dedicated by Governor of California Edmund G. Brown at ceremonies held at the terminal facilities on July 1, 1965. Figure 104 shows the terminal facilities in operation.

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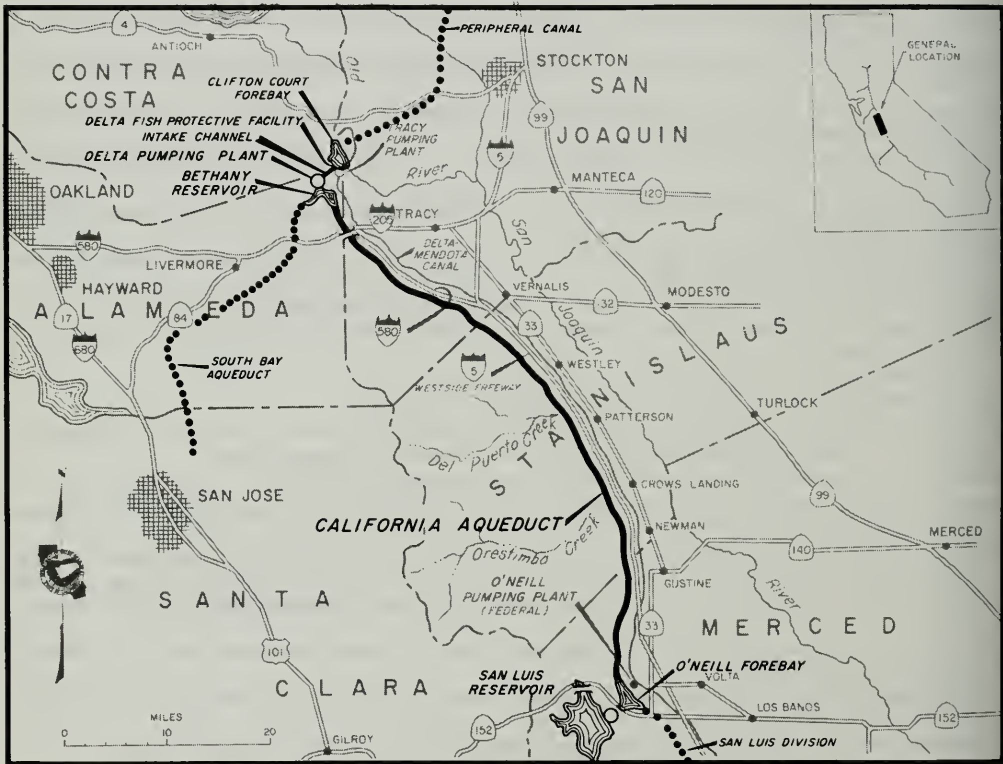


Figure 105. Location Map—North San Joaquin Division

CHAPTER IV. NORTH SAN JOAQUIN DIVISION

Introduction

Role in the State Water Project

The North San Joaquin Division is the beginning of the California Aqueduct, extending from the Sacramento-San Joaquin Delta to its juncture with the San Luis Joint-Use Facilities at O'Neill Forebay. Most of the waters developed by the State Water Project enter the North San Joaquin Division. Design and construction of the conveyance facilities discussed in this chapter took place from 1960–1969.

Hydraulic Function

The North San Joaquin Division was designed to convey 10,000 cubic feet per second (cfs) to O'Neill Forebay and 300 cfs to South Bay Aqueduct. Flow in this division is conveyed mainly by canal section. After the initial lift of 244 feet by the Delta Pumping Plant, water flows by gravity to O'Neill Forebay with a total drop in water surface elevation of 19 feet. Flow is controlled by a series of gated check structures which divide the Aqueduct into 12 pools under the "controlled volume concept" discussed in Chapter I of this volume and Volume V of this bulletin. There are three canal drains in the Division and four small turnouts, with a total delivery capacity by the latter of 48 cfs.

Geography, Topography, and Climate

The North San Joaquin Division is located in Contra Costa, Alameda, San Joaquin, Stanislaus, and Merced Counties, extending along or near the western foothills of the San Joaquin Valley from the Delta southward for a distance of 68.4 miles (Figure 105). The terrain traversed consists of gently rolling hills on the edge of the valley floor.

In its route southward, the Aqueduct generally parallels Interstate Highway 5, a major north-south transportation route, and the Delta-Mendota Canal of the U. S. Bureau of Reclamation.

The California Aqueduct in this division does not pass close to any city. The area is rural and agricultural, with orchards, vineyards, and row crops. Dry farm land and grazing land are interspersed.

The climate is mild in the winter with long, hot, dry summers. The Fahrenheit temperature will range from an average low in the mid-20s during the winter to maximum temperatures slightly above 100 degrees during the summer months. Rainfall along this reach of aqueduct averages about 12 inches per year and occurs primarily during the winter months. Average annual rainfall decreases as the aqueduct alignment trends southward from the Delta and aggregates considerably less than the average annual rainfall that occurs on the western slopes of the Coast Ranges. The streams encountered are ephemeral and rise in the adjacent Coast Ranges. Major streams crossed in the North San Joaquin Division are Del Puerto, Orestimba, and Garzas Creeks.

Floodflows result primarily from general rainstorms but can be caused by occasional isolated thunderstorms in the summer months. General storm runoff is characterized by moderate peak flows, low-to-moderate flood volumes, and durations that last from three to six days.

Features

The North San Joaquin Division originates with Clifton Court Forebay which was formed between the southern Delta channels of Old River, West Canal, and Italian Slough. At present, this is the point where project water enters the distribution system after flowing through Delta channels. The future Peripheral Canal will replace the Delta waterways as the link between the Sacramento River and Clifton Court Forebay.

A 2.7-mile reach of unlined intake channel extends from Clifton Court Forebay southward and, with the Forebay and the Delta Fish Protective Facility, provides the intake facilities to Delta Pumping Plant. Delta Pumping Plant, the only pumping plant for the California Aqueduct within this division, lifts not only the total flow for the California Aqueduct but also that for South Bay Aqueduct. Bethany Reservoir, a small regulating reservoir within the confines of the California Aqueduct, is located 1.2 miles south of Delta Pumping Plant. This reservoir is the forebay for South Bay Pumping Plant and the origin of South Bay Aqueduct. Descriptions of the design and construction practices followed for both forebays (Clifton Court and Bethany) are contained in Volume III, Delta Pumping Plant and South Bay Pumping Plant along with their discharge lines are treated in Volume IV, and Delta Fish Protective Facility is described in Volume VI, all of this bulletin. A statistical summary of North San Joaquin Division conveyance facilities is presented in Table 7.

There are no other regulating or storage reservoirs in the 63.2 miles of concrete-lined canal extending from Delta Pumping Plant to O'Neill Forebay. The Aqueduct was divided for design and construction purposes into the following sections: intake channel, 3.0 miles; Delta Pumping Plant to Chrisman Road, 17.8 miles; Chrisman Road to Del Puerto Canyon Road, 18.5 miles; Del Puerto Canyon Road to Orestimba Creek, 11.7 miles; and Orestimba Creek to O'Neill Forebay, 15.7 miles. The sum of these reaches do not add up to the total division length, because the discharge lines from Delta Pumping Plant and the reach across Clifton Court Forebay are not included.

TABLE 7. Statistical Summary of North San Joaquin Division

INTAKE CHANNEL	
Type	Unlined—trapezoidal
Dimensions	Depth, varies from 30 to 35 feet; bottom width, varies from 80 to 60 feet; side slopes, vary from 3:1 to 1¾:1; length, 2.7 miles
CANAL	
Type	Concrete-lined—trapezoidal—checked
Dimensions	Lined depth, 32.6 feet; bottom width, 40 feet; side slopes, 1½:1; length, 63.2 miles
Capacity	10,300 cubic feet per second, Delta to Bethany Forebay—10,000 cubic feet per second, Bethany Forebay to O'Neill Forebay
Freeboard	2.5 feet lined and a minimum of 2.2 feet of earth berm above lining
Lining	4-inch unreinforced concrete—sealed longitudinal and transverse contraction joints on a maximum of 12½-foot centers
Bridges	46 vehicular—3 railroad—1 pedestrian
Check Structures	12 four-radial-gate structures
Cross-Drainage Structures	32 culverts—47 overchutes—15 drain inlets
Canal Drains	3, one at Corral Hollow Road, one at Del Puerto Creek, and one at Orestimba Creek
SIPHONS	2, one at Orestimba Creek and one at Garzas Creek
OPERATIONS	Manual on-site control or remote control from area control center, Delta Field Division

Geology and Soils

Geology

Part of the North San Joaquin Division is in the foothills of the Coast Ranges which contain sedimentary rocks folded into northwest-trending folds. Much of the mountain flanks along the San Joaquin Valley are on the easterly flank of an anticlinal fold—a situation which results in an easterly dip of the older sedimentary rocks along the canal alignment. In the vicinity of the Aqueduct, the folded sedimentary rocks range from Cretaceous to Pleistocene in age and consist of shale, sandstone, siltstone, and claystone. Where the canal undercuts bedding planes of these easterly dipping rocks, particularly in areas of weak rock, landslides can occur.

Most of the canal alignment goes through younger alluvial deposits on the San Joaquin Valley floor rather than the older, folded, sedimentary rocks in the adjoining hills to the west. The soft alluvial deposits consist of varying mixtures of gravel, sand, clay, and silt.

No major fault systems are crossed by the Aqueduct in the vicinity, so the probability of damage by fault movement seems remote. There is an exposure to earthquakes, however, primarily from those occurring along the San Andreas, Hayward, and Calaveras fault systems approximately 30 miles west of the Aqueduct.

Soils

Soils encountered vary widely within short distances and range from fat clay with swelling characteristics to highly pervious stream gravels. Between these extremes, lean clay, silt, sand, and gravel of various mixtures occur. Thickness of the soils ranges from thin layers on some of the more resistant exposed strata to thick mantles on the valley floor. The streambed soils exhibit typical wave-wash deposition from floodflows and diverse patterns of size gradation.

In general, these alluvial soils can be classed into three groups: (1) residual soils derived from parent material in the immediate area, (2) material deposited in alluvial fans where canyons open onto the valley floor, and (3) stream deposits formed along the major drainage courses. As can be expected with the diversity of soils encountered, the permeability of the soils also was variable, and different degrees of ground water elevations and extent were experienced during excavation.

The fat clays, which generally occur in the dark-colored adobe-type topsoils, potentially were troublesome because of their expansive or swelling capacities. If these clays remain saturated, additional swelling will not occur but, if exposed when dry or semisaturated and subjected to additional moisture, they will expand.

The changing soil conditions also proved troublesome in using local cut material for embankments, and careful analysis and mixing of the material were required to produce the desired permeability and compaction characteristics.

Landslides

During preliminary design of the Aqueduct, it became apparent that the optimum canal route would expose construction to the risk of dip-slope landslides. A degree of exposure to slope failure in cuts has become an accepted risk in present-day, economical, earthwork construction; however, slides must be held to a minimum. The increasing capability of heavy earth-moving equipment to move large volumes of material at relatively low cost assures control of the risks involved.

The variation of rock and soils precludes elimination of all risks of landslides even with an extensive subsurface exploration program. Such a program, however, will expose the majority of potential risks which can and should be eliminated by alignment changes. Treatment of landslides was anticipated by the specifications for the construction contracts and by inclusion of bid items for slide removal and correction.

Design

Eight contracts were employed for the final design and construction of the North San Joaquin Division conveyance facilities. The work performed will be described on a north-to-south basis from the intake channel to O'Neill Forebay, although the work was not undertaken in that order.

Two of the eight contracts included the early construction of high canal embankment sections and the related cross-drainage culverts and structures in Orestimba Creek and Bethany Forebay areas. Those fill sections where the invert of the canal would be above original ground at any point were constructed full length and allowed to consolidate for at least one year prior to placement of concrete canal lining.

The balance of the earthwork, canal structure, and canal lining activities was included in four subsequent contracts. Two contracts were let for construction of the unlined intake channel.

During final design, a study backed by computer analysis of capabilities for certain transient hydraulic conditions was performed by the Department of Water Science and Engineering (formerly the Department of Irrigation) of the University of California at Davis. The study of the portion of aqueduct between Delta Pumping Plant and Bethany Reservoir revealed that for total plant outages of five minutes and for instantaneous outage of one-half of the plant, water surface drawdown in the upper portion of the canal would not exceed approximately 3 feet. The region affected by drawdowns exceeding 2 feet did not extend beyond 2,100 feet downstream from the discharge line outlet structure. The results were within the limiting conditions imposed in the design of Bethany Forebay Dam which permits rapid fluctuation between water surface elevations 243 and 239 feet.

A companion study was made to determine the effects of closing all checks and gates simultaneously within the Division at the design flow rate of 10,000 cfs. Drawdowns under these conditions were found to not exceed 2.5 feet and, for most pools, range from 1 to 2 feet and occur within 45 minutes. The linear extent of 2 feet or more of drawdown was 3,500 feet downstream of the check or gate. The study also revealed that the gates could be overtopped a maximum of 2.1 feet.

As these studies were made for exceptional emergency conditions, the results were considered favorable. Spillage damage, if any, probably would occur only within right-of-way limits and would be confined to minor erosion of embankments or operating roads.

Model Studies

Checks. A series of model tests to develop the best information possible on the head loss through check structures was performed at the hydraulic laboratory of the University of California at Davis. The testing was done for the North San Joaquin Division design program but also applied to subsequent design programs.

Tests were performed on 1:16 scale models of six different design configurations, which included the inlet and outlet transitions and the elevation differentials between the canal and the check inverts.

Since complete dynamic similarity between model and prototype did not exist, the usual approximate scale effect corrections had to be introduced for translation of model data to the prototype. These corrections consisted of separating the boundary friction losses from the form losses in the model, scaling the form losses to prototype size by Froude's criterion, and adding appropriate boundary friction losses to these scaled values to obtain the total prototype losses.

Separate estimates of the boundary friction losses in the inlet and outlet transitions and the rectangular gate section of the model were made by applications of Manning's equation. A value of "n" = 0.011 was selected for the standard tests performed on models with waxed concrete boundaries, on the basis of the Colebrook-White analogy to Nikuradse's pipe friction data. For the estimation of boundary friction in the prototypes, the same combined procedure employed in the models was used. For the prototype, however, a value of 0.016 was adopted for Manning's "n".

In general, the major sources of head loss were the outlet transition and the trailing edges of the piers. The losses in the converging inlet transitions were smaller than expected. The model tests indicated that head loss through the check structure configuration adopted for the North San Joaquin Division (Figure 106) would be 0.0527 of a foot instead of the 0.10 of a foot estimated during early design. The latter figure

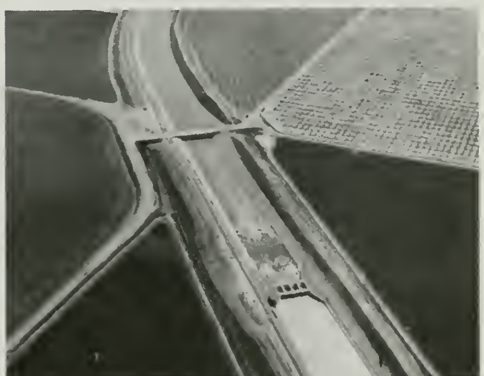


Figure 106. Check Structure

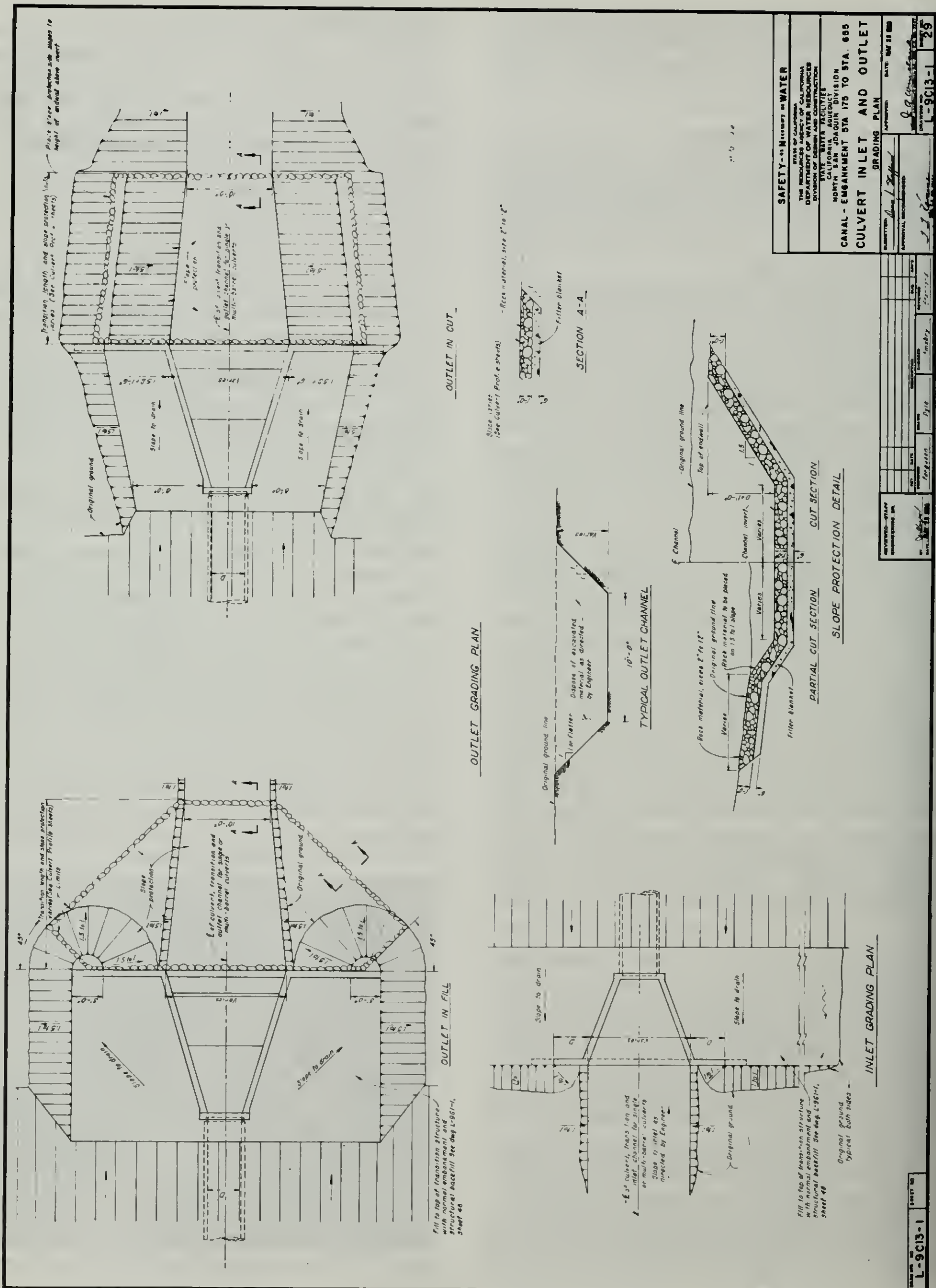


Figure 107. Culvert Inlet and Outlet

had been calculated as 0.5 times the change in velocity head through the check structure, plus the friction loss.

In determining the economics of head loss at the various canal structures, 1 foot of head at Delta Pumping Plant was valued at $450,000.

Culverts. Hydraulic model studies also were conducted on a standard outlet structure to be used in conjunction with the North San Joaquin Division cross-drainage culverts (Figures 107 and 108). The design of the structure was based on concepts developed by the Public Works Department of New South Wales, Australia. Adaptations were incorporated into the design to fit the project conditions and correct deficiencies detected during the model study.

The outlet structure was required to minimize scour at and beyond the culvert outlet with the least tailwater without collecting debris or ponding water in the structure above the culvert inlet. The basic structure configuration was used for both single- and double-barrel culverts. The dimensions of the structure were given as a function of the diameter of the culvert.

A cutoff wall to prevent erosion under the structure extended a minimum of 3 feet below the sill and horizontally across the entire structure including the wingwalls.

Transverse rollers (stationary surface undulations) developed during maximum flow conditions may overtop the sidewalls; therefore, where such conditions existed, the walls were extended vertically 1 foot or the backfill embankment was protected with riprap.

Protection against scour was provided downstream from the structure in cases where the culvert discharged onto a flat bed of erodible material. Usually the protection consisted of riprap or a concrete slab.

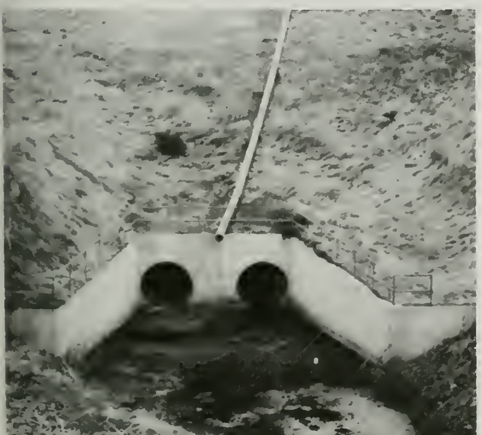


Figure 108. Culvert Outlet

Figure 109. Culvert With Discharge Basin and Canal

The model study also determined the limit of the effective action of the structure based on the velocity in the culvert. Higher velocities were permitted in the larger size culverts than in the smaller sizes. Since the dimensions of the structure were given as a function of the culvert diameter, the total length of the structure was greater with a large-diameter culvert than with a small-diameter culvert. The velocity was reduced to a greater degree before the bed of the outlet drainage channel was reached at the end of the larger structure because of its length.

Culvert Discharge Basins. An overflow discharge basin (Figure 109) for cross-drainage culverts was developed for several locations in lieu of the standard culvert outlet structure and the usual deep outlet channel which extended to daylight. The structure was designed for situations where it was advantageous to eliminate the channel downstream from the culvert outlet. A 2-foot-diameter pipe was used to drain the basin and to carry low flows into the existing drainage channel. High discharges were intended to spill out of the basin at the end of the culvert and continue downslope as sheetflow. The sloping reinforced-concrete sides of the structure provided ease of construction and facilitated cleaning the basin. A model study of the culvert discharge basin was conducted by the University of California at Davis, and the design proved to be a very effective energy dissipator at all flows tested.

Hydraulic Structures

In general, concrete design of canal lining, appurtenances, and hydraulic structures was in accordance with ACI 318-63 and ACI 318-56. Working stress and ultimate strength were based on $f'_c = 3{,}000$ pounds per square inch (psi), $f_c = 1{,}350$ psi, $f_s = 20{,}000$ psi, and $n = 10$ except under special conditions, where concrete strengths of $f'_c = 3,500$ psi and $f_c = 1,575$ psi with $n = 8.5$ were used to reduce the dead load of the larger structures.

A thermal coefficient of expansion per degree Fahrenheit of 0.000006 of an inch per inch and a shrinkage coefficient of 0.0002 of an inch per inch were used for the concrete.

Steel reinforcement was specified as intermediate-grade or hard-grade billet steel in accordance with ASTM Designation A15, deformed, or rail steel ASTM Designation A16.

Design of structural steel was based on "Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings" of the American Institute of Steel Construction.

The modulus of elasticity for all grades of steel was assumed at 29,000,000 psi and a coefficient of expansion of 0.0000065 per degree Fahrenheit.

Steel Pipe

Steel pipelines were designed using ASTM A441 or A36 steel having a yield stress of 30,000 psi, ultimate strength of 50,000 psi, and design stress of 15,000 psi. Buried pipelines are protected with coal-tar enamel and protective wrapping. Exposed piping, such as overchute pipe, was coal-tar-enamel-lined with a protective coating of paint on the outside. Steel pipe was fabricated in accordance with the applicable provisions of AWWA Standards C602 and C205. Bituminous coating and lining conformed to AASHO Designation M190, Type A.

Concrete Pipe

Design and manufacture of reinforced-concrete pipe were in compliance with ASTM Designation C76, Class III. The joints were bell and spigot type having ASTM Designation C361 as minimum requirements. Bends in concrete pipe were made by using beveled-end pipe or precast elbows.

Bridges

The design of reinforced concrete used in vehicular bridges was in accordance with AASHO specifications and supplemented by the California Division of Highways' (now the Department of Transportation) "Bridge Planning and Design Manual". Working stress design was based on $f'_c = 3,000$ psi, $f_c = 1,200$ psi, $f_s = 20,000$ psi, $f'_c = 16,000$ psi, and $n = 10$.

The design of reinforced concrete used in railroad bridges was in accordance with "American Railway Engineering Association" specifications. Working stress design was used for the substructures based on $f'_c = 3,000$ psi, $f_c = 1,350$ psi, $f_s = 20,000$ psi, $f'_s = 16,000$, and $n = 10$; the cast-in-place deck slabs are composite with the prestressed girders and required $f'_c = 4,000$ and 4,500 psi concrete.

Prestressed-concrete girders were used in the superstructures of the 3 railroad bridges and 19 vehicular bridges in the North San Joaquin Division. The design of the prestressed girders was in accordance with AASHO specifications supplemented by Division of Highways' "Bridge Planning and Design Manual". The analysis for prestressed-concrete bridge girders was based upon the elastic theory with design loads at working stress. The girders were designed so that prestressing could be performed by either the pretensioning or the post-tensioning method, the method being at the contractor's option. A wide range of prestressing steel types was allowed. They were: (1) high-tensile wire conforming to ASTM Designation A421, (2) high-tensile wire strand conforming to ASTM Designation A416, and (3) extra-high-strength wire strand.

Embankment and Cut Slope

Slope stability design for canal embankments and excavation utilized both the Fellenius "Swedish Circle" or "Most Dangerous Circle" and the "wedge" analysis. The type of analysis used was dependent on site conditions.

The circle-type analysis prevailed where homogeneous soil conditions existed and where soil formations were bedded in an approximately horizontal manner. Solutions were found both graphically and by means of computer programs. The wedge analysis was used where noncircular failure boundary conditions were indicated.

The factors of safety utilized in the design of embankments and cut slopes for the canal, with the seismic forces assumed to act in the horizontal direction only, were:

Condition	*Minimum Allowable Factor of Safety*
Construction without seismic	1.25
Construction with 0.1g seismic	1.00
Operating without seismic	1.50
Operating with 0.1g seismic	1.20

Canal Excavation and Embankment

Excavation for the canal included excavation for the cut section, overexcavation of cut sections to remove material unsuitable for concrete lining foundation, and overexcavation to remove material unsuitable for embankment foundation.

To prevent excessive seepage or piping through the foundations, the embankment foundation and canal prism were overexcavated to remove cohesionless free-draining material when it occurred adjacent to the canal prism. The overexcavation extended to a relatively impervious foundation or 4 feet below the invert and 10 feet horizontally outside the side slopes. Backfill for these conditions consisted of compacted embankment material.

Where dry expansive clays were encountered, overexcavation was extended 3 feet below the invert and 4 feet below the concrete lining, measured perpendicular to the side slopes. The overexcavated area then was backfilled with a select compacted material.

In areas where the rock type and the angle of the bedding plane indicated a "high potential" for sliding, the potential slide mass was overexcavated to the dip angle of the bedding planes and replaced with compacted material.

Where wet or moist expansive clays were encountered and it was not considered necessary to remove these clays, a 1½-foot layer of unexcavated material measured perpendicular to the surface was left on the surface of the canal prism. This material was kept moist until concrete lining operations when the excess material was removed by trimming machine. This measure was taken to prevent desiccation and subsequent expansion of these clays by leakage when the canal went into operation.

Slope stability studies of the embankments were based on information from the geological exploration program and results of laboratory testing of selected samples.

The canal embankment consisted of two zones: the zone adjacent to the canal called "compacted" embankment, and the outer zone under the operating road called "normal" embankment. Canal embankment was constructed on a foundation cleared of all stumps, large roots, buried organic material, abandoned pipelines, and culverts. The foundation for the normal embankment was scarified 0.5 of a foot deep at 3-foot minimum intervals parallel to the canal prior to placement of normal embankment. The foundation for the compacted embankments was scarified longitudinally a minimum of 0.5 of a foot deep, moisture-conditioned, and compacted.

Compacted embankment material was selected on the basis of the following qualities: workability, compactability, adequate shearing strength, low permeability, and consolidation. The compacted embankment of the fill section was compacted to 95% of relative compaction in 6-inch layers with a width of 7 feet at the top of the lining and side slopes of 1½:1 in the prism and 1:1 within the embankment (Figure 110). The remainder of the fill section and that which supported the operating road was comprised of normal embankment placed in 2-foot layers and compacted by routing equipment over it. Waste material was spilled outside the normal embankment or in spoil areas but was rounded and shaped to blend with the surrounding terrain.

Checks

For the purpose of limiting ground water uplift on the lining, the maximum permissible water surface drop across a check for a zero flow condition was limited to 1.5 feet.

The invert of the Aqueduct was raised 6 feet at the gates of the check structures (Figure 111) for economy in gate and check-structure design. The transitions at the checks increased the average flow velocity by forcing the water inward and upward. The only disadvantage to this upward transition was that a 6-foot-deep pool of water would have to be pumped over the gate sills if the canal was to be dewatered completely. Stoplog grooves were provided at the upstream and downstream ends of each check-structure bay to permit dewatering or isolation of a single gate. Steel stoplogs were designed for full hydrostatic loading on each side of the log. The logs also were interchangeable between check bays.

Check-structure design was based on the following four conditions: (1) one stoplogged bay dewatered at a time with no head differential across the check; (2) canal dewatered downstream, all gates closed, no stoplogs; (3) canal dewatered upstream, all gates closed, no stoplogs; and (4) seismic loading of 0.1g under normal operating conditions.

The radial gates were designed for full hydraulic loading on each side of the gate. The gates were not designed for icing conditions or seismic loading, as they were not expected to encounter these stresses.

Siphons

The length of a siphon undercrossing was determined by providing a flood waterway area over the siphon which would pass the design storm with an average velocity of 10 feet per second. The design storms were assumed to scour the channel over the siphons to obtain the required waterway area. The scour was to be limited by a heavy layer of riprap or other protective measures to protect the siphon.

A standard check structure with four radial gates was located upstream of each siphon (Figure 112), and a standard inlet transition from a trapezoidal to a rectangular section formed the entrance to each check. Each of the four siphon barrels was transitioned from a 20-foot-wide by 23½-foot-high section at the entrance to a 16-foot-square main barrel section. In order to reduce concrete forming costs, the entrance and exit of the siphon were designed with the vertical transition separate from and independent of the horizontal transition.

Structural design of the siphon members was based upon the critical combination of loading cases listed below:

Case 1—Minimum external load only; floodway channel scoured down to the riprap boundary.

Case 2—Maximum external load only; channel filled in with sand and silt to normal streambed elevations.

Case 3—Full water load in siphon with all possible combinations of siphon barrels full or empty. Full water load was assumed with canal water surface at the top of the canal embankment.

The allowable unit stresses under normal conditions were based on the current ACI codes. One-third overstress was allowed for the "dewatered canal" condition.

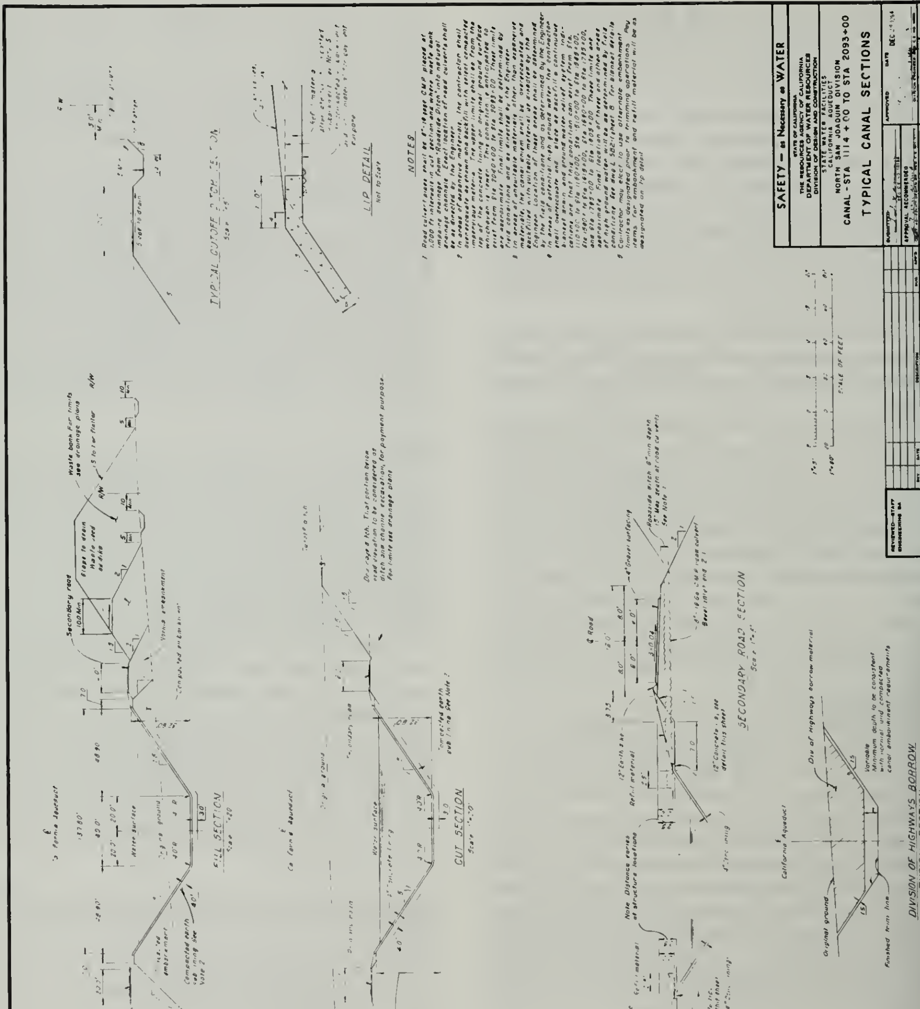


Figure 110. Typical Canal Sections

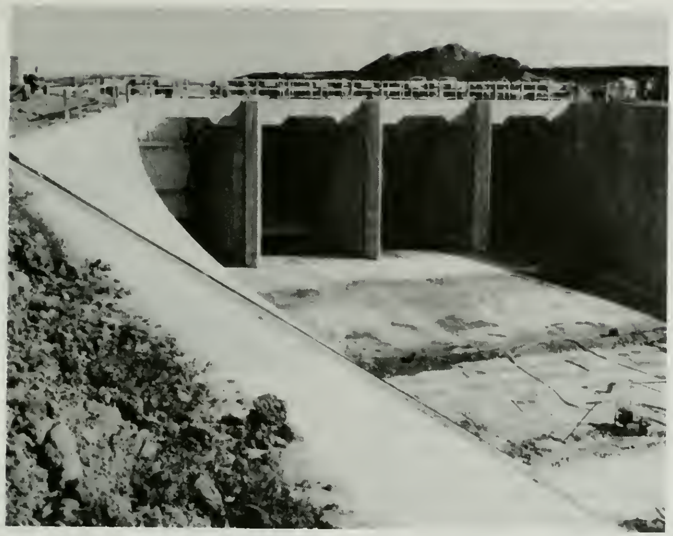


Figure 111. Check Structure Nearing Completion

Figure 112. Check Structure Upstream of Siphon

To prevent flotation of the siphon structure, the full external cover on the siphon ($1\frac{1}{2}$ feet of compacted embankment under 6-inch filter material, 2 feet of riprap, and 5 feet of normal embankment) must be maintained when dewatering any siphon barrel. When necessary to empty more than one siphon barrel, well points or a comparable dewatering system will be required to prevent flotation forces from shifting the siphon on its foundation.

The specifications required the filling of the siphon barrels with water upon completion of construction to prevent flotation prior to operation of the canal.

Drainage Structures

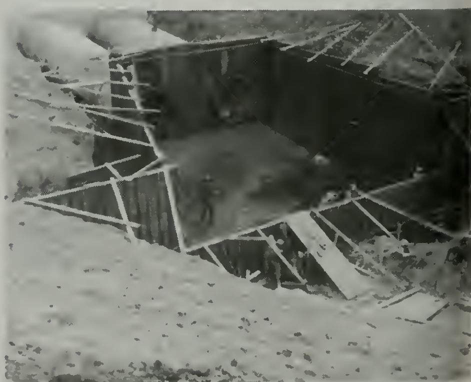
All culverts (Figures 113, 114, 115, and 116) under the canal were constructed of precast reinforced-concrete pipe. Hydraulic design of the culverts considered both open-channel and pressure-flow conditions. The design of culverts utilized flood-routing techniques wherever storage was available at the inlet. Culverts were provided with inlet and outlet transitions (Figure 114), cutoff walls, and energy dissipators. In culvert design, the approach velocity was neglected, and all the velocity head was assumed to be lost at the outlet. The entrance loss coefficient varied with the design and type of culvert inlet.

The reinforced-concrete pipe culverts were designed for earth loadings, external hydrostatic water loads, live load, and dead load of the structure. Because the loading of a culvert under the canal embankment varied significantly along its length, the culvert pipe was designed for as many as six increments of loading to take advantage of savings in concrete and reinforcing steel. The laying lengths of culvert pipe were divided on engineering drawings into zones which required a minimum wall thickness and reinforcing steel area. The inside diameters were held constant.

Earth loading on the culvert was taken as the average weight of soil above the section of pipe under consideration, including increase in weight due to saturation. No reduction in unit weight of soil due to flotation was permitted as the critical loading would occur before this degree of saturation was obtained. The weight of water in the canal prism and traffic load over the embankment was considered as surcharge loading. Stability of the structure against flotation was checked wherever the possibility existed that the culvert may be empty and the surrounding ground saturated.

Internal water loads and bursting pressure loads generally were neglected for culverts under high fills since this increase in design stress is small and loading was of short duration.

Reinforced-concrete flume overchutes and welded-steel pipe overchutes were used to convey cross-drainage flows over the Aqueduct. It was found that a rein-



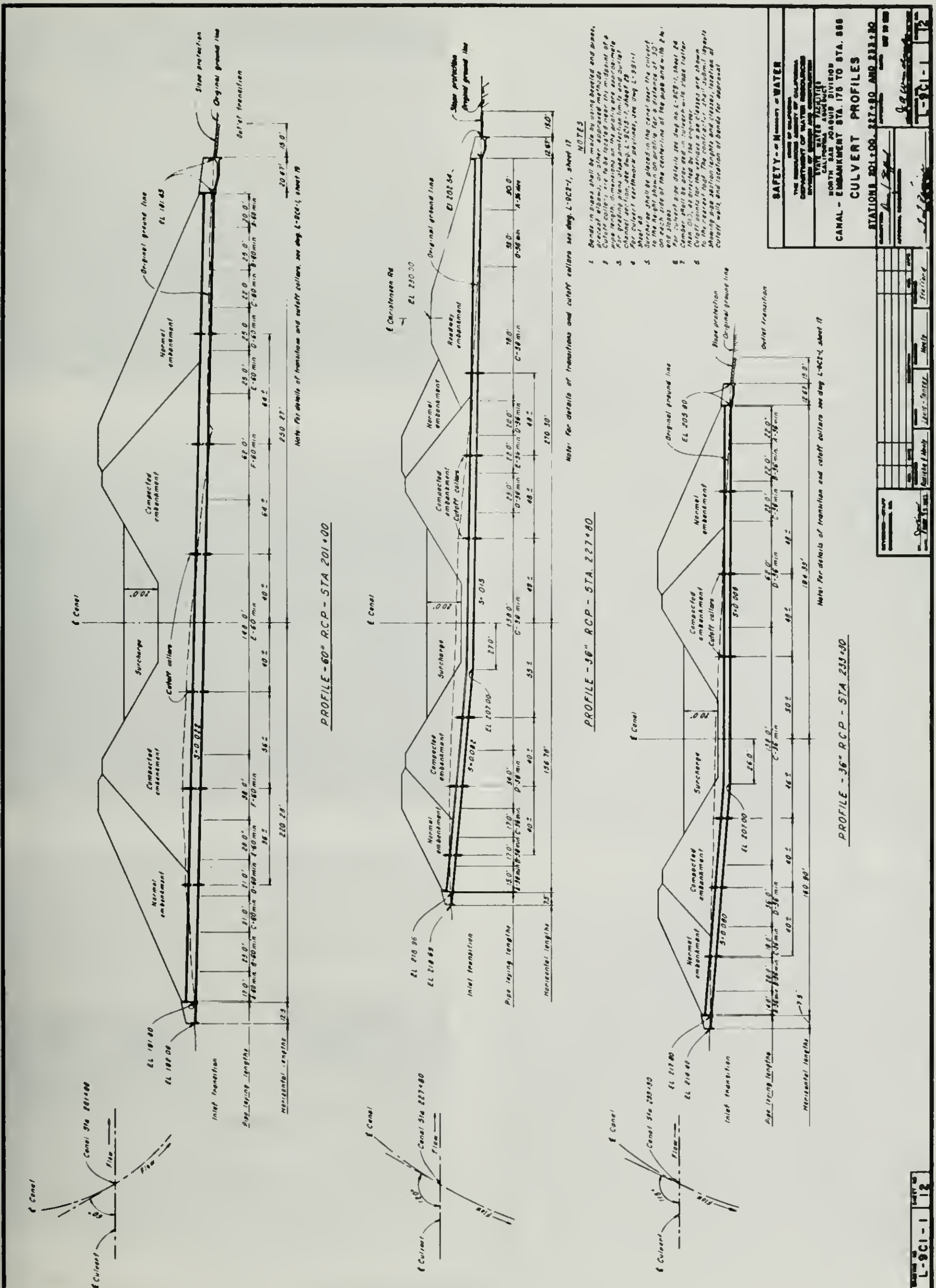


Figure 116. Culvert Profiles

forced-concrete flume overchute was more economical to use than a welded-steel pipe when the required size of pipe exceeded 36 inches in diameter.

Hydraulic design for pipe overchutes was similar to hydraulic design for a culvert. Pipe overchutes (Figures 117 and 118) consisted of a reinforced-concrete inlet transition, corrugated-metal pipe under the operating roads, two spans of welded-steel pipe across the canal supported by an abutment pier on the ends and a center pier in the canal, an outlet transition, and an energy dissipator.

The reinforced-concrete overchute (Figure 119) consists of the inlet transition, culvert section under the roads, flume, outlet transition, and energy dissipator. The flume was designed to pass the routed flow of the 100-year frequency storm with plus 0.5 of a foot of freeboard, either in the subcritical or supercritical state. To be conservative in computed freeboard, an "n" of 0.014 was used in Manning's equation for determination of depth of flow in the flume. For a conservative estimate of the energy to be dissipated, an "n" of 0.012 was used to determine the velocity of the flow entering the energy dissipator. Stilling basins or baffled aprons were utilized for energy dissipation.

Drain inlets, either gravity or pumped, were used in lieu of conveying flows across the canal where only a small quantity of acceptable quality drainage water was involved. The poor quality and the excessive silt load of irrigation made it imperative to limit the use of drain inlets. Eight-inch, corrugated-metal-pipe, gravity, drain inlets into the Aqueduct were used to drain the operating road ditches in lined canal cut sections or where long, continuous, high, waste banks exist. Corrugated-metal pipe downdrains with an energy dissipator were used to convey flows downslope from one channel to another. All culverts under roads and dikes were of corrugated-metal construction.

Subsurface Drainage System

In areas of high ground water, the canal lining was underlain with a 5-inch filter blanket (Figure 120) and drainage collector system. The drainage water is collected in sumps at convenient locations for removal. Permanent pump installations were used in some locations with automatic controls. At other locations, portable pumps will drain the sumps. Whenever the canal was located in pervious material which was divided by an impervious stratum, a complete ground water relief system was installed with the filter zones completely penetrating the impervious layers. This system was necessary to prevent irrigation water from becoming entrapped by the strata and damaging the canal lining by means of uplift pressure. Both asbestos cement and clay drain pipe were specified for this underdrainage.

Figure 117. 36-Inch Steel-Pipe Overchute

Figure 118. 36-Inch Steel-Pipe Overchute Energy Dissipator

Figure 119. Quinto Creek Overchute

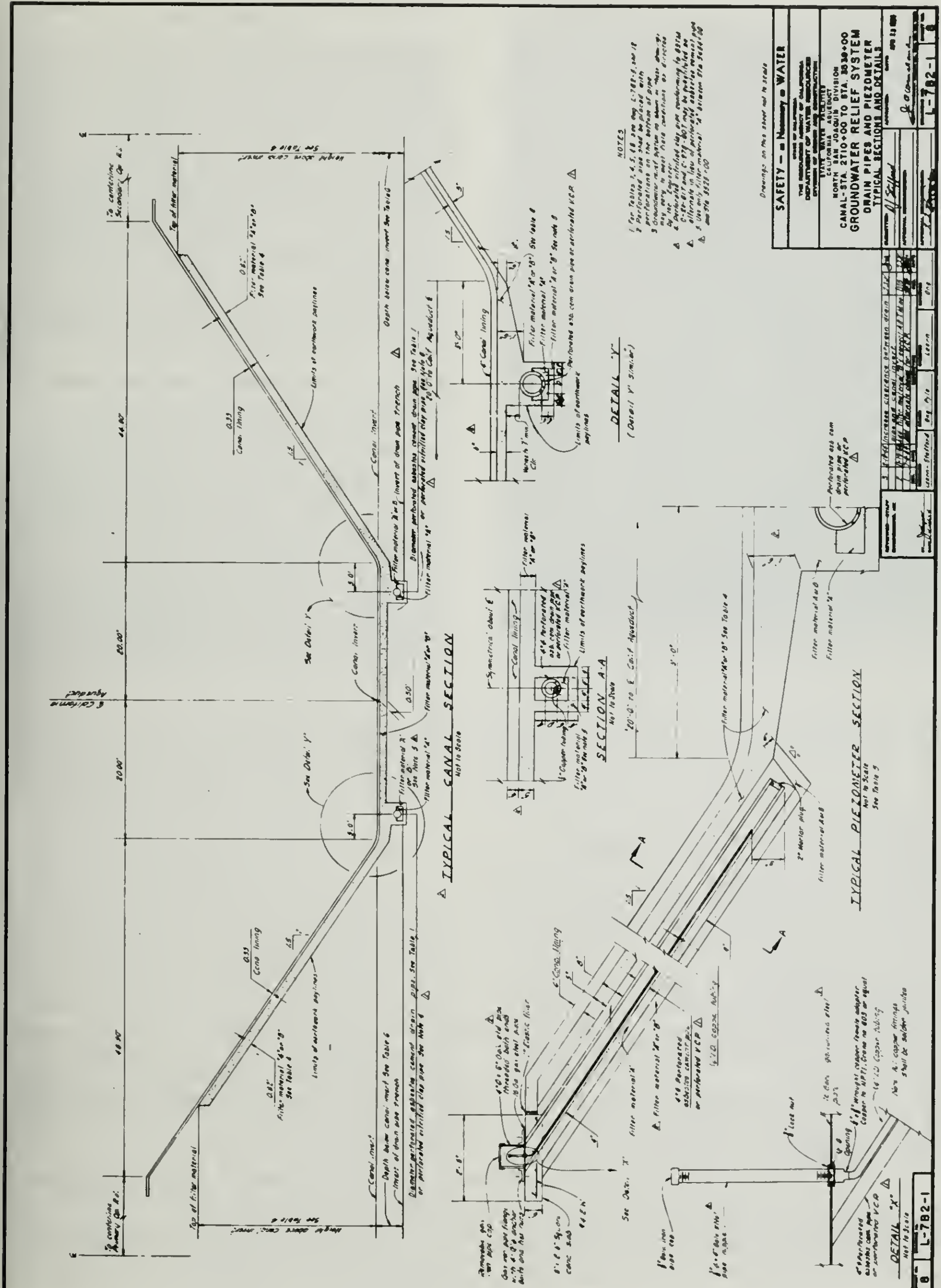


Figure 120. Ground Water Relief System

Figure 121. Intake Channel Connected to Clifton Court Forebay

Construction

Construction supervision was administered by the project office at Patterson, California. Temporary offices during initial construction were located at Delta Pumping Plant, Tracy, and Gustine. Field trailers generally were located in the same vicinity as the headquarters location for the contractor's personnel.

Field laboratories were established near the contract work sites at Delta Pumping Plant, Vernalis, and Gustine. Water quality laboratory work generally was done in Sacramento, and concrete laboratory work was accomplished both at the field laboratories and in Sacramento.

General information about the major contracts for the construction of the facilities for this division is shown in Table 8.

Design and Construction by Contract Reaches

Intake Channel to Delta Pumping Plant

Design. The intake channel alignment was designed initially to connect an interim Delta diversion point on Italian Slough with Delta Pumping Plant. Subsequently, the intake channel and Clifton Court Forebay were connected, eliminating the interim diversion on Italian Slough (Figure 121).

The Delta Pumping Plant location and the intake channel termination were determined by comparing the cost of deep channel excavation with the cost of discharge lines plus excavation and backfill. The re-

TABLE 8. Major Contracts—North San Joaquin Division

	Specification	Low bid amount	Final contract cost	Total cost—change orders	Starting date	Completion date	Prime contractor
Intake Channel Mile 0 to Mile 1.6	65-07	$2,448,718	$3,251,922	$557,773	5/ 3/65	2/23/67	Piombo Construction Co.
Intake Channel Mile 1.6 to Mile 3.3	63-22	5,699,280	5,955,761	370,177	7/27/63	10/26/64	Peter Kiewit Sons' Co.
Canal Embankment Mile 3.3 to Mile 12.4	63-28	1,657,542	1,962,933	111,335	9/ 6/63	11/20/64	Morrison-Knudsen Co., Inc.
Aqueduct—Delta Pumping Plant to Chrisman Road Mile 3.3 to Mile 21.1	65-06	11,145,713	13,489,551	755,665	6/ 3/65	2/28/68	Wunderlich Company
Aqueduct—Chrisman Road to Del Puerto Canyon Road Mile 21.1 to Mile 39.6	65-08	12,390,374	13,557,940	851,594	5/20/65	12/22/67	Western Contracting Corporation
Canal Embankment Crow Creek to Garzas Creek Mile 47.3 to Mile 56.8	63-01	750,117	885,469	43,637	2/ 8/63	12/ 5/63	Eugene Luhr & Co. and Elmer G. Wendt, Inc.
Aqueduct—Del Puerto Canyon Road to Orestimba Creek Mile 39.6 to Mile 51.3	64-02	7,775,441	10,920,847	1,320,232	5/11/64	12/ 8/67	Western Contracting Corporation
Aqueduct—Orestimba Creek to O'Neill Forebay Mile 51.3 to Mile 67.0	64-29	12,694,128	21,127,465	6,787,527	9/ 3/64	1/31/68	Western Contracting Corporation

sulting depth of cut at the Pumping Plant was approximately 160 feet.

Construction. Two contracts were utilized for the intake channel construction because of the difference in type of work involved. Deep excavation was included in the first contract, "Intake Channel, Mile 1.6 to Mile 3.3", to allow sufficient time to excavate the pumping plant bowl. The relocation of Bruns Road, two oil pipeline crossings, and a 78-inch pipe flume for the Byron-Bethany Irrigation District canal crossing were included in the contract.

The intake channel prism was varied by design to a deeper and narrower section at the Pumping Plant to minimize the deep rock excavation. The side slopes, which varied from 3:1 to 1¾:1, were protected from wave wash by a 12-inch layer of riprap extending from elevation −5.0 feet to +7.0 feet.

The second contract, "Intake Channel, Mile 0 to Mile 1.6", included compacted canal embankments and channel excavation, a Southern Pacific Railroad bridge, Byron Road bridge, Clifton Court Road bridge, and irrigation facility relocations.

The Byron-Bethany Irrigation District later requested that the Department construct a temporary earth ditch around the end of the cut for the first contract in lieu of the steel pipe flume which had been designed to carry their canal over the intake channel. Later, by agreement, the District constructed a temporary steel pipe siphon through the intake channel. The siphon was removed when the District completed construction of a small pumping plant on each bank of the intake channel to supply their severed distribution system.

The intake channel of the California Aqueduct was constructed in five contracts between Italian Slough and Delta Pumping Plant.

In chronological order, they were as follows:

1. Intake Channel, Mile 1.6 to Mile 3.3
2. Intake Channel, Mile 0.0 to Mile 1.6
3. Fish Protective Facility
4. Intake Channel Modifications
5. Clifton Court Forebay

The Fish Protective Facility, intake channel modifications, and Clifton Court Forebay are covered in Volumes VI and III of this bulletin. Only the intake channel is discussed in this volume.

Construction of the intake channel, Mile 1.6 to Mile 3.3, began in August 1963 and was completed in October 1964. The work consisted of dewatering the construction site, excavating the intake channel, rough excavating for Delta Pumping Plant bowl, constructing roads and a bypass for an irrigation lateral, and providing support piers for oil and gas pipeline crossings.

The contract provided an item for dewatering the construction site. The contractor intercepted subsurface water in the intake channel excavation with ditches and sumps and pumped the water from the sumps into a main drainage ditch from the site to Italian Slough, a distance of approximately 2½ miles.

The contractor excavated about 12,000,000 cubic yards of material using twin-engined 40-cubic-yard scrapers on the steeper slopes and single-engine 40-cubic-yard scrapers on the less steep slopes. Excavation was performed on a two-shift basis for most of the job.

Horizontal drains were installed in the excavated cut slopes to reduce possible slides. About 13,232 lineal feet of drilling for the drains was accomplished in 99 drain locations. Under direction of the owners, both Standard Pacific and Standard Oil Company pipelines were relocated.

One county road, Bruns Avenue, was relocated and various other access roads were constructed to the operation and maintenance area and Delta Pumping Plant. Operating roads along the intake channel were constructed as channel excavation progressed.

Construction of the intake channel, Mile 0.0 to Mile 1.6, began in May 1965 and was completed in February 1967. The majority of the work on this reach involved excavation of about 2,500,000 cubic yards of the intake channel and construction of channel embankments from Italian Slough to Mile 1.6. Physically, this work was divided by the Southern Pacific Railroad and an adjacent county road, Byron Road. The Railroad and Byron Road were relocated on detours to allow for excavation and construction of bridges. After bridges and approaches were completed, the Railroad and Byron Road were relocated back to final alignment and the detours removed.

Between Italian Slough and the Railroad, there was a major problem of providing dry working conditions. Water was pumped from the excavation area into ditches leading to a ponding area, adjacent to Italian Slough, from where it was pumped into Italian Slough.

Generally, the earthwork operation method was to excavate the channel and haul the material to designated waste areas or to embankment areas, depending on the quality of material. Rubber-tired scrapers were used in dry areas and tractor-drawn scrapers in damp and wet areas. In extremely wet areas, draglines loaded muddy materials into assorted types of hauling equipment.

There was a shortage of suitable material for compacted embankment. Good material was found at the site of the Delta Fish Protective Facility. Most of the embankments were constructed with material borrowed from that area.

A temporary wooden bridge was constructed over the mouth of the intake channel on Clifton Court Road. A county agreement provided for removal of the bridge 10 years after its completion.

Delta Pumping Plant to Chrisman Road

Design. This portion of the Aqueduct was divided into two contracts. The first, which covered roughly one-half of the 17.8-mile-long reach, was for early con-

Figure 122. California Aqueduct Below Bethany Reservoir Outlet Structure

Figure 123. Corral Hollow Culvert

Figure 124. Corral Hollow Culvert Outlet

struction of high fill sections, two bridges and associated road relocations, miscellaneous drainage structures, utility relocations, and relocation of Bethany Forebay spillway. The intent of this early contract was to allow at least one year for settlement and consolidation of the higher embankments before the canal lining was placed.

The second contract included the completion of the canal prism; all finishing earthwork and canal lining; and the remaining drainage structures, bridges, and miscellaneous structures. The second contract also contained the outlet structure (Figure 122) for Bethany Reservoir (see Volume III of this bulletin) and the outlet structure for Delta Pumping Plant discharge lines (see Volume IV of this bulletin).

Major features of the reach were 16.4 miles of concrete-lined canal; 3 county road bridges; 6 farm bridges; 2 railroad bridges—one each for the Southern Pacific and Western Pacific Railroads; Delta Pumping Plant discharge lines outlet structure; 4 aqueduct check structures; and 41 cross-drainage structures, including one major structure—the Corral Hollow multibarrel box culvert. In addition, there were 14 reinforced-concrete pipe culverts, 5 reinforced-concrete box flume overchutes, and 10 welded-steel pipe overchutes.

The canal prism was designed using the Colebrook-White equation assuming an equivalent sand roughness of 0.005 feet. This corresponds to an "n" value of 0.016 in Manning's formula for a design flow rate of 10,000 cfs. The lined canal cross section, which is constant throughout the North San Joaquin Division, has a bottom width of 40 feet, water depth of 30 feet, side slopes of $1\frac{1}{2}$:1, and a top width of 137.8 feet (Figure 110). The southward slope is 0.24 of a foot per mile with a 4-foot-per-second velocity at design discharge. Bend losses were considered insignificant in themselves with a bend radius of five times the canal width at the water surface as the minimum radius of curvature.

The aqueduct lining, freeboard, berms, drainage structures, operation roads, bridges, seepage, and evaporation losses were in agreement with the general design criteria covered in Chapter I of this volume. The Division of Highways designed and constructed bridges for U.S. Highway 50 and Patterson Pass Road overcrossings. Through an interagency agreement, they also excavated 4,032,000 cubic yards of excess material from the canal prism for Interstate 5 embankment

Corral Hollow culvert (Figure 123), the major drainage structure in the contract, provides west-to-east flow of local runoff under the Aqueduct adjacent to the Corral Hollow Road overcrossing. The culvert was designed as a reinforced-concrete box with three barrels, each having a height of 7 feet and a width of 11 feet. The inlet to the culvert was designed as an open, reinforced-concrete, flare transition from open stream channel to rectangular culvert. A 24-inch canal drain was incorporated in the structure to permit dewatering of the canal by gravity flow.

The inlet ends of the culvert barrels are 12 feet high. A vertical straight-line transition to the nominal 7-foot height was made in the initial descending 100 feet of barrel to prevent excessive depth of ponding upstream.

An 80-foot by 40-foot stilling basin with 10 feet of depth below the culvert barrel invert was designed for energy dissipation at the outlet (Figure 124). Attaining a hydraulic jump in this structure was assured by large dentations in the chute and basin and a raised outlet sill. The 29-foot-high basin walls were designed to be free-standing.

An open upward transition to the outlet channel invert was made commencing at the sill. The outlet channel invert at the head end is level with the culvert barrel outlet invert. The area around the basin and for 50 feet downstream of the sill was heavily riprapped to prevent undermining of the basin. The outlet drainage channel was constructed 2,200 feet long with an 80-foot base width and 2:1 side slopes.

Construction—First Contract. Work on the first contract, "Canal Embankment, Mile 3.3 to Mile 12.4", began in September 1963 and was completed in November 1964. In addition to the embankment and drainage structures, the work included bridges at Grant Line and Christensen Roads and the relocation of portions of these roads as well as Midway Road.

Much of the construction effort in this contract was devoted to placing 13 reinforced-concrete pipe culverts ranging in size from 36 to 90 inches in diameter. Mobile cranes were used to place the pipes. Laborers with pneumatic and mechanical hand tampers compacted backfill around the pipe, cutoff collars, and inlet and outlet transitions. This work was supervised closely to ensure that leakage from the canal would not have a passageway along the culvert.

Canal excavation was by pneumatic-tired scrapers. The full prism was excavated leaving 18 inches of cover over the lining foundation for final trimming.

At the north end of this reach, moderate-to-heavy ripping was required in the Panoche formation before excavation could take place. Some blasting of hard boulder concretions, not uncommon in this formation, also was required.

The Panoche formation contains weathered jointed shales interbedded with the more resistant rocks. These shales part easily along bedding planes, which resulted in several wedge-type slides when the beds were undercut. The bedding dipped from 10 to 50 degrees toward the canal prism.

A slide in canal excavation at the Christensen Road crossing resulted in moving the centerline of the bridge 75 feet upstream and adjusting the road alignment to that of the new bridge site. Otherwise, the overcrossing was constructed without difficulty.

The Neroly formation encountered in the center portion of this reach required only minor ripping and, with effort, broke down sufficiently for use in fills. In the vicinity of the Highway 50 overcrossing, some sliding occurred along bedding planes within clay beds.

The Tulare formation, occurring in the southern portion of the reach, was composed primarily of claystone and siltstone. The more plastic of the interbedded clays tended to air slake and exhibited expansive properties.

Expansive clays were removed in lieu of leaving 18 inches of cover over the lining foundation. The expansive clays were overexcavated 4 feet normal to the side slopes and 3 feet below invert elevation and were replaced with selected and compacted material. Some of the expansive clays which were saturated had little potential for further expansion and were left in place with 18 inches of cover for moisture retention.

The proper blend of embankment materials was obtained by intermixing, disking, and blading prior to compaction. Two tractor-drawn sheepsfoot rollers and a self-contained double-axle sheepsfoot were used for the majority of this work.

Construction—Second Contract. Work on the second contract, "Earthwork, Concrete Lining and Structures, Delta Pumping Plant to Chrisman Road", began in June 1965 and was completed in February 1968.

The contractor's schedule provided for construction of the Southern Pacific and Western Pacific Railroad "shooflys" as the first order of work and for canal excavation to progress generally from north to south. Several months' delay occurred in concluding agreements between the railroad companies and the Department. The contractor found it necessary, because of this delay, to readjust his schedule and bypass the railroad crossing sites to provide a sufficient section of finished canal prism to permit steady placement of the concrete lining.

The excavation and embankment operations in the southern half of the contract, which was mostly in the Tulare formation, had less difficulty with slides than the first contract. At the height of the earth-moving period of the contract, over 20 earth movers plus supporting equipment were being used. To facilitate the earth moving, the contractor left earth-plug roadways in the canal prism about every 800 feet. Later, these plugs were removed, usually by dragline prior to trimming operations.

Some minor sliding or slumping occurred where joints were undercut by the canal excavation. The largest of these minor slides was about 12,000 cubic yards.

A major slide, requiring 340,000 cubic yards of excavation, occurred in the vicinity of the Highway 50 overcrossing in the Neroly formation. The failure occurred along a clay seam. The material was removed to a depth of 20 feet below invert elevation and was replaced with 240,000 cubic yards of compacted embankment.

The contractor set up a batching plant at Patterson Pass Road. Construction of the two railroad bridges, Corral Hollow Bridge, and drainage and check structures with transitions was started as soon as excavation in those areas was completed.

A pile test at the Western Pacific Railroad bridge site indicated low resistance, so the piles for both railroad bridges were extended an additional 10 feet for the abutments and 20 feet for the piers. The piers for other bridges and for overchutes were constructed after the canal lining was placed. Piles, where required, were driven before lining placement. After placement, concrete lining over the piles was removed by sawing to permit construction of the footing. If the top of the footing occurred at invert grade, a construction joint was incorporated with the footing. If the elevation was below the invert, the joint was incorporated with the pier structure.

Steel forms were used for the check-structure walls and counterforts and for the Corral Hollow Bridge. Plywood panels were used on all other bridge piers and overchute piers. Since many of the piers were of the same dimensions except for height, reusable plywood forms proved a valuable aid.

Movable forms were used on the Corral Hollow drainage structure. The bottom slab was placed first; then wheel-mounted forms for the side walls and top slab of the barrels were jacked into place. After the concrete was placed and had gained sufficient strength, the forms were wheeled ahead for the next placement.

Concrete was moved from the batch plant in agitator trucks and placed by bucket, chutes, or belt. Slump was controlled during transportation in the hot months by adding ice to the mix.

The contractor used new earth trimmer and concrete slip form paver machines for this contract (Figure 125). Trimming was on one-half of the canal prism for each pass of the equipment. The slip form paver and finishing jumbos would pave one-half of the canal prism, less a 4-foot center section, on the initial pass. The paver's return run with the aid of a special, 8-foot, movable, form insert completed the lining.

The trimmer was equipped with a "bucket line" that trimmed earth to grade and carried the excavated material to a conveyor belt at the upper end of the trimmer. The conveyor belt then carried the material to the operating road, where it was either shaped into a line or grade for the operating road or was removed with scrapers. On the initial pass, the trimmer was held to line and grade by sensors running on guide wires set near the canal centerline and the top of lining hinge point. On the return pass, the invert grade was taken from the previously placed lining.

The concrete aggregate was dry-batched at a central plant and hauled in double-trailer bottom-dump trucks to the tilting mixers, which were set up at con-

Figure 125. Paving Train Placing Concrete Canal Lining

venient locations along the canal side. The cement, aggregate, pozzolan, water-reducing admixture, air-entraining agent, and water were added at the mixer in 8-cubic-yard batches. The mixed concrete then was transported to the slip paver. A conveyor belt on the paver deposited the concrete at the leading edge of the paver, where it was vibrated just ahead of the slip form.

The paver was equipped to form both the transverse and longitudinal contraction joints. The transverse joints were made by first cutting the fresh concrete with a vibrating groove cutter, floating the joint with a vibrating float, and trimming with a specially shaped tool developed by the contractor. The contractor received permission to use "Constop", an embedded-type polyvinyl chloride waterstop, in the longitudinal joints. This material mounted on reels (Figure 126) was fed into the concrete by guides attached to the leading edge of the slip form paver and, as the paver passed over the concrete, the longitudinal joints were formed and filled with the waterstop.

The finishing jumbo following behind the paver permitted a hand-troweled finish to be given to the entire surface. In addition, any rock pockets were repaired with grout and a vibratory trowel for consolidation. A curing jumbo followed closely behind the finishing jumbo and applied a surface seal by spraying a membrane curing compound. A distributor truck moving along the canal bank kept the jumbo supplied with curing compound.

The transverse joints were sealed with a two-compound polysulfide polymer. The material was mechanically extruded into the joints by a machine which operated on a jumbo which spanned one-half of the canal prism at a time. The transverse joints were first sandblasted and then air-cleaned just prior to applying the sealant. Contact was required by the polysulfide sealant with the longitudinal waterstop whenever the transverse joint crossed over the longitudinal joint.

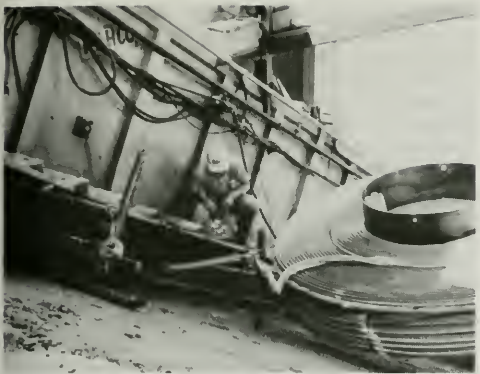


Figure 126. Waterstop Being Placed From Reel on Paving Jumbo

Whenever the lining operations reached an in-place bridge, it was necessary to disassemble the equipment train, transport it to the other side of the bridge, and reassemble the equipment, which required about two days. Some hand concreting was required in these locations; however, most of the lining under bridges and adjacent to check structures was placed in a checkerboard arrangement of 12-foot panels to allow for curing. These panels were placed from a special slip form screed and were considered a "hand work" operation.

Random cracks occurred in some of the lining and were sufficient to warrant repair. Random cracks were those which occurred at places other than at contraction joints. The causes of random cracking were: (1) contraction cracks parallel to and alongside the longitudinal contraction joints where the "Con-stop" failed to form a weakened section, (2) plastic shrinkage cracks which occurred as the concrete hardened, (3) pressure cracks caused by water pressure behind the lining or from expansive material under the lining, (4) loading cracks caused by the movement of vehicles or machinery on the lining, and (5) shrinkage cracks which occurred from unknown causes. The cracks, if over 0.025 of an inch in width, were repaired by chipping to provide a ⅜-inch-wide by ⅝-inch-deep groove, which was filled with polysulfide sealant. In some instances, where the cracks were the result of swelling subgrade material, enough lining was removed to permit replacement of the swelling material before relining. A total of 13,729 lineal feet of cracks was repaired.

Chrisman Road to Del Puerto Canyon Road

Design. This 18.5-mile reach of the California Aqueduct was designed and constructed as a single contract, which included both earthwork and canal lining, as well as bridges, drainage features, and appurtenances. The reach is located farther out on the valley floor than the section to the north and, therefore, was less susceptible to cut slope failures caused by undercutting of sedimentary beds in the foothills. Alluvial clays, silts, sands, and gravels were encountered throughout most of the reach except near the downstream end where fissured claystones of the Oro Loma formation occur.

This heavily traveled reach required a large number of road crossings. During construction, portions of the canal prism were excavated by the Division of Highways for use as fill material in construction of Interstate 5 adjacent to the Aqueduct.

The major construction features of this section were: 18.5 miles of lined canal; 9 county road bridges; 4 private road bridges; 3 check structures with radial gates and operating appurtenances; and 19 cross-drainage structures consisting of 8 reinforced-concrete flume overchutes, 6 welded-steel pipe overchutes, 4 reinforced-concrete pipe culverts, and 1 cast-in-place culvert. Five water courses—Lone Tree, Hospital, Ingram, Kern, and Del Puerto Creeks—drain the area west of the Aqueduct with Ingram and Del Puerto Creeks providing the larger of the potential flows. The Del Puerto Creek culvert structure also incorporates a canal drain for the Aqueduct.

The canal and appurtenant structures were similar in design to the contract reach previously described. However, there were two features sufficiently different to warrant discussion. These items are the drainage structure for Del Puerto Creek at the southern end of the reach and the Hetch Hetchy Aqueduct crossing at Blewett Road in the northern portion of the reach.

Del Puerto Creek culvert (Figure 127) was designed to safely pass the 500-year frequency floodflows of Del Puerto Creek. This creek crosses the California Aqueduct 900 feet downstream of its culvert undercrossing of adjacent Interstate 5.

Figure 127. Del Puerto Creek Culvert

Runoff from the 73-square-mile drainage basin was computed to have peak flows of 8,300 cfs and 12,400 cfs, respectively, for the 100-year and 500-year design flood hydrographs. Discharges of these magnitudes would form an unregulated storage pool upstream (west) of the California Aqueduct and another pool upstream from Interstate 5. Normal runoff flows would traverse the two facilities without ponding. Routing the 500-year design flood hydrograph through the two culverts was computed to cause ponding uphill of the canal embankment to elevation 238 feet with 8,500 cfs discharged downslope of the canal. The embankment crest, elevation 238.43 feet, would not be overtopped. The pool formed upstream of the state highway fill would be high but sufficient freeboard existed to prevent overtopping of the highway.

As constructed, floodflows from the upstream drainage area enter the first ponding pool. From this point, they pass through a 22-foot, cast-in-place, concrete, arch-type culvert under the Interstate 5 fill section before being stored temporarily in the second pool between Interstate 5 and the California Aqueduct. The waters then pass under the aqueduct embankments through a reinforced-concrete culvert structure. The culvert consists of a flared inlet with reinforced-concrete floors and side walls; a downward-sloping, 16-foot-inside-diameter, reinforced-concrete, culvert barrel; and a rectangular stilling basin outlet structure.

Beyond the barrel, the flow will spread and descend in the flared "U"-shaped chute into the stilling basin. Most of the excess energy will be dissipated in a hydraulic jump which will be contained completely in the stilling basin. One row of baffles was provided in the stilling basin to induce maximum energy dissipation in the shortest possible distance. Basin walls were designed to be of sufficient height to contain the hydraulic jump for the 100-year maximum routed discharge with 5 feet of freeboard and without any freeboad for the 500-year routed discharge.

Elimination of erosion downstream of the stilling basin was assured by riprapping the section of the 5:1 upward slope of the unlined outlet channel and the partial conical-shaped section forming the swirl basin outside the free-standing stilling basin walls.

A 24-inch, slide-gated, canal drain (Figure 128) was incorporated into the Del Puerto Creek culvert for the purpose of dewatering the canal by gravity.

The Hetch Hetchy Aqueduct, owned and operated by the City and County of San Francisco, crosses the California Aqueduct alignment with three pipelines: (1) a 66-inch, wrapped, steel pipe; (2) a 61-inch reinforced-concrete pipe; and (3) a 79½-inch, wrapped, steel pipe. These were relocated across the Aqueduct as wrapped steel pipe of the same inside diameter supported on a single reinforced-concrete pier at the center of the aqueduct prism and pile abutments at the edges of the prism.

Two maintenance bridges were designed to carry the load of a 50-ton crane lifting a 27-ton load. The two bridges were positioned to serve the three relocated pipelines and a possible future pipeline. The center pier, which supported pipes and bridges, was designed oversize to accommodate a future pipeline.

Access to the crossing was from either the primary or secondary operating roads, or from Blewett Road, which is located about 50 feet downstream of the pipelines. Relocation of the pipelines over the canal prism also required that the pipeline grade be raised. An air release and vacuum valve was placed on the high spot of each line. A drain facility was provided to pump water from the sag created west of the canal when the pipeline was raised. An access manhole was included for each pipe on the east side of the canal. Structural steel and weld design for the crossing was based on IASC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, April 1963.

Construction. This contract, "Earthwork, Concrete Lining and Structures, Chrisman Road to Del Puerto Canyon Road, Mile 21.1 to Mile 39.6", began in May 1965 and was completed in December 1967.

The contractor for this reach also was the contractor for the two reaches extending south to O'Neill Forebay. As these other reaches were under construction during the time of this contract, the contractor could and did move men and equipment between the three reaches to best meet his work schedules.

A landslide which occurred nearby in an adjacent contract reach prompted additional exploration stability analysis of six cut slopes at the south end of the contract reach. As a result, the canal centerline was realigned 75 feet to the east, or downhill, to avoid potential trouble. This relocation was for approximately 5,000 feet just south of Del Puerto Creek in the Oro Loma formation.

The contractor requested and received permission to irrigate the excavation prism prior to excavation operations. In a nine-week period in the fall of 1965, 52,700,000 gallons of water was applied to the excavation areas. Where known to exist, overlying fat clays were removed before watering to allow better penetration of the water. Actual canal excavation was delayed until five months later to allow for penetration of the water.

Expansive clays in the upper 4 to 6 feet of the canal prism behind the future concrete lining were excavated by scrapers and replaced with compacted embankment. Embankment and rough excavated surfaces, which would receive concrete lining, were left 18 inches high and this material removed to grade by the trimming machine.

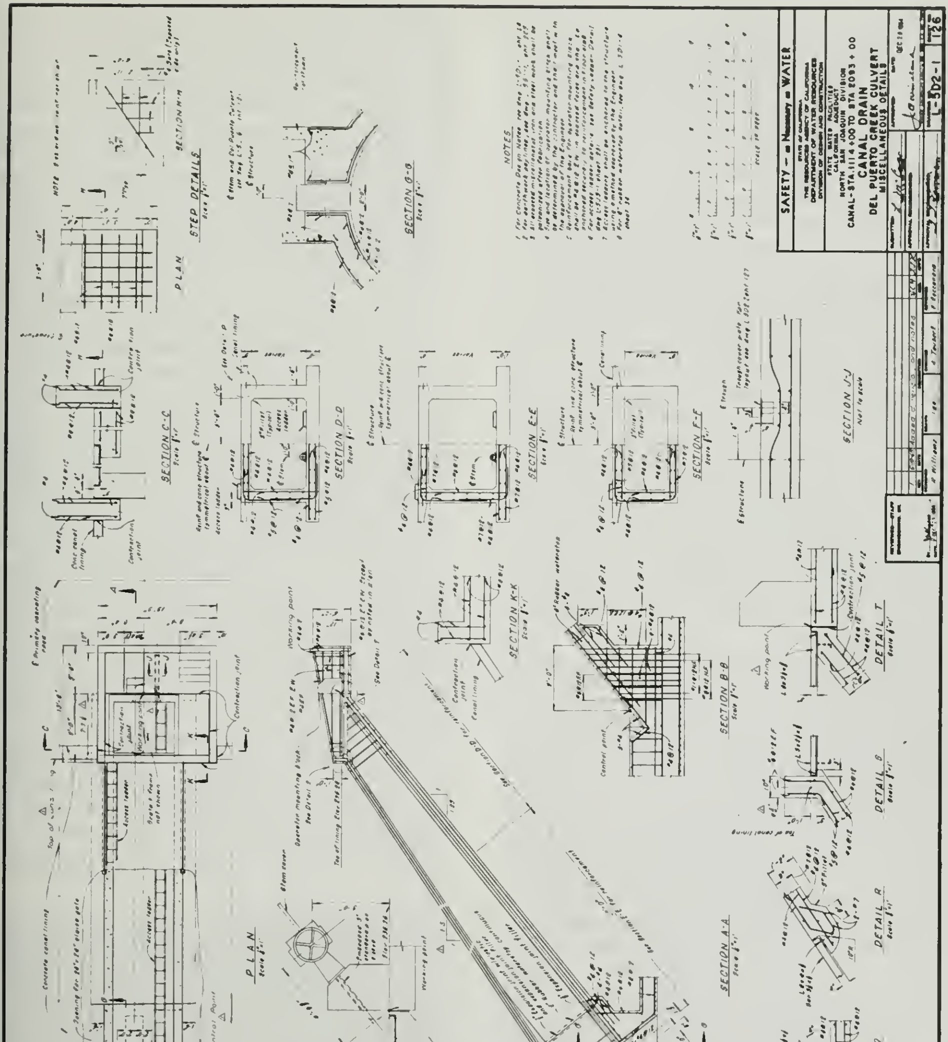


Figure 128. Del Puerta Creek Canal Drain

Two draglines with 14-cubic-yard buckets (Figure 129) performed much of the excavation and piled the excavated material in the normal embankment section. The use of draglines resulted in little down time because of wet weather conditions. After trimming and lining, tractor dozers and scrapers shifted and shaped the operating roads to grade and shaped the remaining waste piles.

Structures which required piers to overcross the Aqueduct were constructed after lining operations, with piles and/or footings being placed prior to lining. This procedure avoided any interference with lining operations. The Del Puerto Road bridge at the south end of the contract was an exception to this procedure since it was necessary to complete this roadway at an early time. This bridge was constructed as part of the adjacent Interstate 5 contract.

Although specific underdrains were not designated, the plans and specifications provided for underdrains at five locations totaling about 3 miles if, during construction, high ground water required this relief. Actually, none of the expected locations required the underdrains; however, they were required at two other locations. One of these was near the Gaffery Road overcrossing, where a perched water table was encountered which required 1,000 feet of underdrain. The other occurred in the vicinity of the Ingram Creek overchute, where about 1,000 feet of underdrain was constructed.

The sequence for the construction of the underdrains was as follows: (1) after rough excavation of the canal prism, the invert drain pipe trenches were excavated; (2) the 4-inch perforated drain pipe was installed, backfilled with coarse aggregate and filter material, and compacted with power tampers; (3) structural excavation was performed for a 36-inch reinforced-concrete pipe pump sump; (4) the pump sump was installed and connections made with the drain pipes; (5) the area around the pump sump and connecting pipe trenches was backfilled with select material and compacted by power tampers; (6) finish-grade trimming was done with a trimming machine modified to place the filter material blanket in conjunction with the trimming operation; (7) filter material on the slope was rolled with an 8-foot by 10-foot smooth-drum roller drawn up the slope by a truck-mounted crane, and invert filter material was compacted by a smooth-drum vibratory roller drawn by a tractor; and (8) concrete lining was placed over the underdrain system.

Movable forms were used in the Del Puerto Creek cast-in-place culvert. The bottom slab was placed first, and then movable forms were installed for the circular culvert side and top wall. Concrete then was placed and, after the concrete had gained the proper strength, the forms were lowered and moved ahead to the next sections to be constructed.

The general sequence of construction for each check structure was as follows: (1) fine-grade the earth foundation for floor slab and transverse beams; (2) place concrete in floor slab; (3) grade, form, and place concrete in the inlet counterfort slab and footings and transition invert slab; (4) form and place concrete in exterior counterforts and wall on one side of centerline; (5) form and place concrete in exterior counterforts and wall on opposite side of centerline; (6) form and place concrete in left, center, and right interior walls; (7) form and place concrete in inlet transition walls; (8) form and place concrete in outlet counterforts and transition walls; (9) form and place concrete in hoist platform; (10) form and place concrete in trunnion beams; and (11) form and place concrete in inlet sand trap and outlet transition.

The concrete lining train performed excellently since the same equipment had been used previously on the two adjacent southern contracts and, consequently, was well job-tested and modified. The train consisted of a trimmer, paver, and two finishing jumbos. Concrete was delivered to the paver in 10-cubic-yard batches carried from the batching plant in agitator trucks. The lining operation was similar to that described in the previous contract but with production rates nearly doubled.

A method was used for the transverse joint which was different from that described in the previous contract in that a semirigid plastic insert was placed in the joint immediately after this joint was struck by a knife on the paver. The insert was left in place to preserve the joint until the sealant was placed. This method proved to be an improvement compared to forming the joint with a vibrating cutter and sandblasting before adding sealant.

The contractor received permission to use Constop for the longitudinal joints conventionally installed from reels mounted in front of the paver. The contractor also asked for permission to use Constop for the transverse joints. He received conditional approval provided that a good seal was obtained where the transverse joint crossed the longitudinal joint. In fol-

Figure 129. Dragline Excavation of the Aqueduct

lowing this method, both Constop joints were placed at the paving machine before any finishing of the concrete to determine the effectiveness of the contact between the two joints. Tests did not reveal a good contact; consequently, this method was discontinued and the plastic insert and sealant method for the transverse joint was reinstated. When applying the sealant at the intersection of the contraction joints, a short section of the longitudinal Constop material was snipped out before applying the sealant. This method provided a better seal than that which required the sealant to only make contact with the Constop at these locations.

Del Puerto Canyon Road to Orestimba Creek

Design. Design studies for this 11.7-mile reach of the California Aqueduct resulted in the preparation of plans and specifications for two contracts. The initial contract was for placement of embankment at three locations where high fills were required to permit at least one year for settlement before lining operations. The fills in this section were as high as 35 feet, with cuts running to a maximum of 70 feet. This initial construction contract also contained similar work in the reach extending from Orestimba Creek to O'Neill Forebay.

This reach of canal extends along the west margin of the San Joaquin Valley and eastern foothills of the Diablo Range. The alignment encounters rolling hills of folded, Tertiary, sedimentary rocks and flat valley areas covered with alluvial clays, sands, and gravels. Tertiary sedimentary rocks in the Kreyenhagen, Poverty Flat, and Neroly formations did not make good compacted embankment material and therefore were used for normal embankment, drain fills, road embankments, or channel ditches.

Ground water required underdrains with a collection and disposal system. The areas of expected high ground water were in the vicinity of Little Salado and Orestimba Creeks. Two types of filter material were specified for the underdrainage: Type A, a coarse material designed with grading characteristics sufficient to handle large flows of water but which would not prevent infiltration of fines, and Type B, a finer material than Type A which would not handle high volumes of water but would resist any substantial infiltration of fines. Type A filter material was used around the perforated pickup pipes to prevent infiltration into the pipes. Type B material was placed around the coarse material and next to the native soils to prevent infiltration of fines.

Figure 130. Aqueduct Excavation With Scraper

No major drainage structures or unusual features occurred in this reach, and the design of the facilities was in accordance with previous descriptions. Major features of this section of the Aqueduct were: 11.7 miles of concrete-lined canal; 4 county road bridges; 4 farm operational bridges; 2 project operational bridges; 2 check structures; and 13 cross-drainage structures, including 3 reinforced-concrete flume overchutes, 9 reinforced-concrete culverts, and 1 welded-pipe overchute. In addition, the Division of Highways designed and constructed an overcrossing of the Aqueduct for Interstate 5 between the Simon Newman and Mays Road overcrossings.

Construction—First Contract. Work on the first contract, "First Stage Construction of California Aqueduct, Station 2500 to Station 3000 (Mile 47.3 to Mile 56.8)", began in February 1963 and was completed in December 1963. The portion of the contract located in this reach consisted of three high embankments between Crow and Orestimba Creeks and attendant drainage facilities. Late heavy rains in April and May caused the contractor to suspend operations for three weeks. During the hot summer period, it was necessary to add ice to concrete mixes and place concrete during very early hours (3 to 5 a.m.) to avoid high temperatures in the mix.

The majority of the embankments were constructed of material from canal excavation, with some material from borrow areas. Excavation was by conventional scrapers with little ripping required. In stratified areas, the scrapers were loaded on a 4:1 slope to provide a better mix of materials. Considerable disking and blending were required to provide suitable embankment material.

Two reinforced-concrete pipe culverts were constructed without incident in this portion of the contract. Structural concrete was delivered by transit mix trucks from a local commercial company.

Construction—Second Contract. Work on the second contract, "Earthwork, Concrete Lining and Structures, Del Puerto Canyon Road to Orestimba Creek, Mile 39.6 to Mile 51.3", began in May 1964 and was completed in December 1967.

The contractor worked only in light excavation areas during the first six months, using small, conventional, earth-moving equipment. As there also were heavy cuts in this contract, a new earth-moving machine was developed. The earth mover was (Figure 130) a three-engine two-bowl scraper with a rated capacity of 80 cubic yards. The capacity in excavation was approximately 65 cubic yards heaped. The first weeks of use of this new machine largely were experimental and a training period for the operators. After about a month, a major structural failure occurred in the equipment, necessitating a shutdown and an extensive equipment reinforcement program. Machines that had been delivered were modified in the field and put back into operation.

Two other large earth movers, a two-engine single-bowl scraper and a three-engine two-bowl scraper with 40- and 28-cubic-yard rated capacities, also were used with very little down time for repairs. One disadvantage of using such large-capacity machines was the difficulty experienced in spreading the material in compactable lifts. This resulted in a high number of compaction test failures and subsequent recompaction of the material.

Ground water in the area north of Little Solado Creek proved more troublesome than expected. Ground water was encountered about 10 feet above invert grade. The contractor first tried dewatering by constructing two sumps below invert grade. Several weeks of pumping from these sumps did not remove enough water, so a well point system was installed. Two-inch well points were spaced on 6-foot centers, 35 feet on each side of the aqueduct centerline. This method proved satisfactory even though the well points interfered with the movements of the earth-moving equipment.

In the vicinity of Mays Road, a side hill cut in the Neroly formation, opened during the first contract, experienced a rotational-type failure on the left side of the prism involving about 100 cubic yards of material. This cut probably had perched ground water as a contributing factor to the slide. Therefore, in addition to the slide correction, an underdrain system was included for the right cut slope. The original slide continued to expand with similar slide action incipient for about 1,000 feet along a weak, fissured, clay seam. Remedial action consisted of overexcavation and compacted refill of the entire canal prism to a maximum of 30 feet horizontally and 10 feet below invert to remove the weak clay material. The top 16 feet of the right side refill consisted of permeable silt, sand, and gravels to permit the underdrain to bleed off any pressure which might occur from drawdowns during operation of the Aqueduct.

At the southern end of the contract, near Orestimba Creek, the ground water level was near invert elevation for approximately 800 feet. Within this area, compacted earth lining was used in lieu of an underdrain system. Streambed gravels in Orestimba Creek were overexcavated 10 feet horizontally and down to the Kreyenhagen formation, which was relatively impervious at this location, or to 5 feet below invert. In addition to the lining, a perforated drain leading to a sump was placed at the bottom of the overexcavation.

On April 30, 1965, with the rough canal excavation down to within 7 feet of grade, a massive earth slide involving about 150,000 cubic yards moved into the right prism excavation. The slide occurred just downstream from the Neils Hansen bridge site and has been called the Patterson slide, as a reference to the closest town.

The first indication of ground movement was rather severe raveling of the sands and gravels exposed in the right cut slope. Within one hour, the toe of the slide mass had moved 60 feet across the bottom of the partially completed excavation and essentially stabilized against the left prism cut slope. Observers reported that the 400-foot-long by 12-foot-high slide scarp opened up in the first 20 minutes. For practical purposes, the failure was immediate and unexpected. An oil pipeline was ruptured by the slide.

This slide occurred where gravels and sands of the Tulare formation overlie the clays, claystones, and sandstones of the Oro Loma formation. At the head of the landslide, 300 feet right of centerline, the beds dipped 40 degrees east but at the canal the beds were nearly flat, forming a critical "slip-circle" configuration. Interstate 5, under construction, was situated immediately west of and above the slide.

Corrective measures taken to stabilize the Patterson slide consisted of:

1. The slide mass immediately adjacent to the canal was excavated several feet into undisturbed material and backfilled with compacted embankment to form a toe key.
2. Part of the slide material at the head of the slide was removed to reduce the driving force. Coordination with the Interstate 5 contractor working at a higher elevation resulted in agreement to accelerate construction in this area, which removed additional material at the head of the slide.
3. The oil pipeline was relocated into the undisturbed formation below the slip plane.

This slide also triggered further investigation of 12 scheduled cuts with similar geologic conditions. Of the 12, 6 were considered unstable enough to require corrective action. Three of the other six locations were avoided by moving the centerline of the canal 75 feet downslope. The remaining three locations were corrected by removing material from the tops of the cut slopes.

During initial lining operations, the contractor solved several procedural problems before a satisfactory lining was obtained. One problem was poor concrete consolidation at the polyvinyl waterstop (Constop) used in the longitudinal joints. This was remedied by installing immersion-type vibrators on each side of each strip being placed. However, about 10,000 feet of initial waterstop installation was found to be so unsatisfactory that an additional groove had to be cut directly above the Constop and filled with the same sealant used for the transverse joints. At the same time, a new and successful procedure was instituted; subsequently, the transverse joints were sealed as soon as the lining had sufficient set rather than waiting until much later when the reach was about to be completed. Special mobile work platforms were developed from which the sealant was applied.

Two check structures were built with little difficulty using both wooden and steel forms. A special flexible slip form was developed for placing concrete in the warped walls. Radial gates were fabricated and installed by separate subcontractors. Some difficulty was encountered with the anchor bolts being out of alignment with the holes in the hinge base, which required the hinge base to be reworked. Also, the contractor was prevented from installing the side seals prior to the gate alignment check to permit correct installation and proper alignment of both the gates and seals.

Prestressed-concrete girders were used for the eight bridges, and some of the girders had excessive bows (in the horizontal plane) of up to 4½ inches. Some of the bows were reduced or eliminated by jacking and pulling against adjacent girders. An investigation showed the bows were attributed to creep in the girder after tensioning and before lateral support was applied by diaphragms. The bows were found to be insignificant in their effect on the structural soundness of the completed structure.

Overchute construction in this reach was the first undertaken in the North San Joaquin Division. A difficulty experienced with the initial structure, which was formed and cast full length, was the opening of the construction joint at the piers and abutments when shoring was removed. These openings, due to beam deflection, then were made half as large by constructing the spans and removing forms one span at a time. The openings required that the joints be sealed from the inside with a cold-applied elastic sealant after form removal.

Orestimba Creek to O'Neill Forebay

Design. Design work in this 15.7-mile reach culminated in issuing of plans and specifications for two contracts: the first for early construction of high fill sections and the second for contract completion.

The canal alignment crossed rolling hills of folded Tertiary and Cretaceous sedimentary rocks that partially are covered with alluvial deposits. The older sedimentary rocks are claystones, siltstones, shales, and sandstones, all of varying hardness. The alluvial soils are varying mixtures of gravel, sand, clay, and silt. Extensive reaches of high ground water were present.

The main features constructed were: 16.5 miles of lined aqueduct; 4 county road crossings; 5 farm operational bridges; 4 department operational bridges; 1 access road; 4 check structures; the terminal structure to O'Neill Forebay; 2 siphon aqueduct undercrossings at Garzas and Orestimba Creeks with a canal drain connection at the latter siphon; and 20 cross-drainage structures, including 11 reinforced-concrete flume overchutes, 4 welded-streel pipe overchutes, and 5 reinforced-concrete culverts.

There were four distinctive features included in the design of this section of the Aqueduct; otherwise, design of the remaining features was in accordance with previous descriptions. Those features distinctive to this reach were: the siphon undercrossings at Orestimba and Garzas Creeks, the acoustical velocity flowmeter and foot bridge just upstream of the outlet structure to O'Neill Forebay, and the outlet structure itself.

In connection with a program of hydraulic model testing of check structures by the University of California at Davis, a model test was made of the O'Neill

Figure 131. O'Neill Forebay Inlet

Forebay inlet structure (Figure 131). This test assessed the energy dissipation and flow characteristics of discharging 10,000 cfs from a typical canal check structure through an outlet transition at head differentials varying between 4 and 8 feet through the gates. The effectiveness of the stilling action and flow characteristics were checked in the vicinity of the gates, outlet transition, riprapped transition, and unlined forebay inlet channel.

It was found that the structure would function satisfactorily under normal conditions only when all radial gates were open the same amount. The baffle piers included in the design at the bottom of the outlet transition did not appear to serve a significantly useful purpose as, in their absence, nonscouring velocities occurred along the entire boundary of the riprapped and unlined sections of the channel.

The radial gates of the O'Neill Forebay inlet structure are 2 feet higher than typical check gates. The structure and radial gates were designed for the following hydrostatic conditions: (1) an upstream water surface at elevation 225.8 feet with no water on the downstream side, and (2) a downstream water surface at elevation 228.0 feet with no water on the upstream side. The downstream transition and the 200 feet of riprapped channel were designed to dissipate the energy of an 8-foot drop in water surface at 10,000 cfs. The bottom slab of the transition at about the two-thirds point along its 175-foot length has a jet deflection block. This block extends over the entire width of the invert except for five 4-foot breaks to provide drainage. An underdrain system was provided for the walls and base slab of the outlet transition to prevent uplift.

From an economic and operating standpoint, it is necessary to know the volume of project water which enters the San Luis Joint-Use Facilities. An acoustical velocity flowmeter was provided in a long straight run of canal just upstream of O'Neill Forebay for instantaneous and volumetric flow measurement. The flowmeter never operated successfully because the mechanism for moving its transducer was clogged with silt and sand. It is still inoperable but is being modified to utilize fixed transducers.

A footbridge (Figure 132) was included to provide cross-canal access to the acoustical velocity flowmeter installation and to serve as a platform from which aqueduct flows can be measured with current meters or similar devices. This bridge was designed without piers in the aqueduct prism to assure the best possible flow conditions for metering purposes.

The Aqueduct was siphoned under Orestimba (Figure 133) and Garzas Creeks with inverted, reinforced-concrete, box siphons 331 feet and 258 feet long, respectively. The siphon design was described earlier in this chapter. A canal drain was incorporated in the Orestimba Creek crossing. This is the southernmost drain in the Division. Unlike the Corral Hollow and Del Puerto Creek canal drains which have slide-gate outlets incorporated into cross-drainage culverts, the Orestimba drain is controlled by a butterfly valve. The valve controls the flow in a reinforced-concrete pipe, which extends from the canal upstream of the inlet transition of the siphon to the Orestimba Creek stream channel downstream of the siphon.

Construction—First Contract. The first work in this reach was done under the contract "First Stage Construction of the California Aqueduct, Station 2500 to Station 3000, (Mile 47.3 to Mile 56.8)". The work was accomplished between February and December 1963 and consisted of the high embankments between Shell and McDowell Roads plus the relocation of Pete Miller Road.

The canal excavation and embankment work was uneventful. About 25,000 yards of high sulfate soils were removed at the downstream end of the reach where the prism was overexcavated 12 feet horizontally and to a depth of from 3 to 8 feet below invert. There were no difficulties from slides.

Ground water was encountered in placing the culverts under the fills which required perforated pipe and gravel underdrains below the culvert inverts for dewatering the excavation. Except at one location, which was left open, these underdrains were plugged with gunite to permit the ground water level to return to preconstruction level. The exception was where the ground water level normally was above the invert elevation of the canal prism and the underdrain for the culvert would complement the future underdrain system in the Aqueduct.

Ground water encountered at the culvert near the McDowell bridge site contained a high concentration of sulfates. A change order was issued which required precast concrete pipe used in the culvert to be manufactured with Type V cement, resistant to attack by sulphates rather than the specified Type II cement.

Construction—Second Contract. Work on the second contract, "Earthwork, Concrete Lining and Structures, Orestimba Creek to San Luis Forebay, Mile 51.3 to Mile 67.0", began in September 1964 and was completed in January 1968.

Single- and double-bowl scrapers and a 14-cubic-yard dragline were used for canal excavation. An 80-cubic-yard double-bowl scraper was structurally strengthened and worked satisfactorily in relatively good material where there was sufficient turning room after load discharge. Load spreading continued to be a problem with large equipment when uneven and oversized layers of deposition required added disking and blending prior to compaction.

Slides proved to be a problem. Four of the more important landslides were: the large Indian Rock slide (Figure 134) just south of Orestimba Creek in the Neroly formation, a large slide just downstream of the McDowell Road bridge site in the Kreyenhagen formation, a slide in the Upper Moreno formation midway between the Cottonwood and Butts Road bridges sites, and a slide in the Lower Moreno formation near Romero Creek at the southern end of the contract.

Several months prior to the initial failure in the Indian Rock area, when excavation of the 120-foot-deep cut had reached a depth of about 40 feet, a 4-inch-thick, sheared, clay seam was exposed in the right cut slope. Intermittent seepage was noted along the seam and, when saturated, the highly plastic clay could be gouged and remolded with light finger pressure. The weak clay seam was conformable to the bedding, dipped 7 degrees northeast (into the excavation), had a strike of 40 degrees with the canal alignment, and represented a potential failure plane of major dimensions.

This cut slope was placed under close surveillance for cracking or other signs which might indicate imminent failure. When the excavation was about 32 feet above final grade, the cut slope on the right side failed and approximately 300,000 cubic yards of material moved in on a 700-foot-wide front.

Sixteen core holes were drilled to locate the clay seam farther to the south where it appeared to extend beneath the existing excavation. All 16 holes encountered the failure plane. Thickness of the seam ranged from a fraction of an inch up to 12 inches, and its continuity was established to a depth well below the invert of the Aqueduct. Three bulldozer trenches were used to expose the seam in the bottom of the excavation. Testing verified that the clay seam was highly plastic. A high montmorillonite content was indicated by X-ray analysis.

Remedial action at Indian Rock consisted of: (1) removal of the weak, sheared, clay seam and all overlying material for a distance of 250 feet right of the aqueduct centerline; (2) laying the right cut slope back to a 4:1 slope; (3) excavating a key at the toe of the right cut slope for support by the backfill; and (4) rebuilding the overexcavated prism.

As the remedial excavation proceeded and an increasing length of the clay seam was exposed at the toe, the slope began to show signs of distress similar to those shown just prior to the initial slide. Finally, with the clay seam daylighted for about 1,000 feet, the redesigned 4:1 slope failed, and the overlying mass again moved downslope on the low-dipping clay seam. A pressure ridge developed near the toe of the slope, where the seam disappeared below the bottom of the existing excavation, for about 500 feet to where the seam was about 9 feet below the bottom of the existing excavation.

A reevaluation of conditions following the unloading caused by the second slide indicated the original remedial plan was adequate and no further change was required. Material was removed from the toe of the slide until temporary stability was reached and design grades obtained, at which point the right side of the prism was rebuilt. With the prism protected, the slope was allowed to continue to move until stability was attained.

Figure 132. Steel Truss Footbridge for Flow Measurements

Figure 133. Orestimba Creek Siphon and Check Under Construction

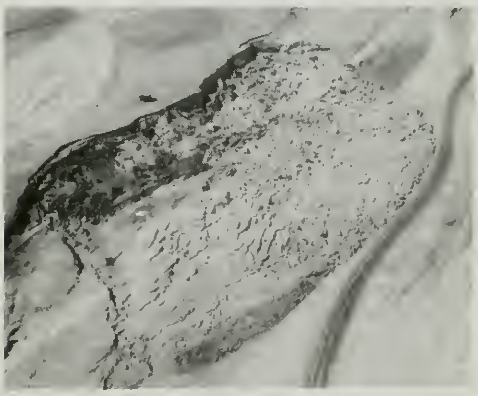


Figure 134. Indian Rock Slide During Aqueduct Excavation

In late November 1965, a large slide occurred in the Kreyenhagen formation sediments near the McDowell Road bridge. The failure was along one or more jointed and weak shale beds which dipped into the excavation from the right side at about 7 degrees. Because similar adverse conditions were known to be prevalent in ground adjacent to that which had failed, this reach was mapped in detail and placed under surveillance. Additional areas of cracking were noted and one other impending failure was recognized.

Corrective work consisted of overexcavation and backfill with competent material. All Kreyenhagen formation materials were wasted with refill material obtained from overlying terrace materials.

During design, potential sliding in the Upper Moreno formation was recognized because of the adversely dipping shale beds. The extent of corrective measures was to be determined after exposure. A few weeks after initial exposure and before excavation was completed, substantial bedding plane slides in this formation developed in the right prism of the Aqueduct between Cottonwood and Butts Roads.

Detailed mapping disclosed geologic conditions responsible for these slides generally were representative of the right prism slope throughout the Upper Moreno formation. The right cut slope along this exposure was redesigned to prevent future operational difficulties. During the remedial work, which consisted of overexcavating a continuous 30-foot-wide section to 7 feet below invert, additional slides occurred in the overexcavated back-slope. All sliding material was removed and refilled with select material.

One-dimensional swell tests of recompacted samples of the overexcavated Moreno formation shales indicated they were expansive. As a result, the shales when excavated were used only in the outer 20 feet of backfill away from the lining. Select alluvial and terrace gravelly soils were placed adjacent to the aqueduct prism. Several weeks after the rough canal excavation had been completed, two rotational-type slope failures occurred in Lower Moreno formation in the right prism slopes near Romero Creek. The bedded, weak, clay shales and claystones had a dip of from 30 to 70 degrees, which was conducive to bedding plane sliding. The shales and claystones also were potentially expansive, as evidenced by rapid deterioration from alternate wetting and drying and rather deep shrinkage cracking. Atterberg limits were considerably higher than those obtained in the Upper Moreno formation. Damp seepage areas in the slopes and invert of the excavation suggested that pore pressures also could have contributed to the slides.

Although rotational failures were possible on either canal slope through this exposure, geologic information was not conclusive enough to justify extensive overexcavation wherever this formation was encountered. Therefore, the correction adopted was to overexcavate approximately 12 feet horizontally on both sides and 3 feet below invert and remove potentially expansive clay shales, thus providing a more stable subgrade material. The two rotational failures were overexcavated a minimum of 10 feet beyond the observed cracking. Select, alluvial, clay, gravel backfill was obtained from a suitable borrow area south of Romero Creek. In addition, a continuous pipe drain was installed below the canal invert on the right side to relieve pore pressures outside of the lining prior to canal filling and during operational drawdowns.

Due to the slides, overexcavation, backfill operations, and required changes, the construction schedule was not met. Accordingly, all work was accelerated to permit water delivery to O'Neill Forebay by October 31, 1967. This acceleration required additional personnel, equipment, and longer working days. One of the methods used to accelerate the work was the substitution of precast or prestressed girders in lieu of cast-in-place girders for the bridges.

Ground water required a large portion of the contract reach to be provided with underdrain protection. Filter blanket usually was applied by the trimming jumbo prior to lining operations. Drain lines were installed during foundation preparation. When equipment was still active in these areas, damage to the underdrain pipes frequently occurred and required pipe replacement. The trimming machine also caused pipe damage because, when traveling its lower track, it operated directly above these pipes.

During the wet winter of 1966–67, all major creeks began to flow. At Quinto Creek, a large pond developed to the west of the empty paved canal for about 2,300 feet. The ponded water entered the highly permeable, uppermost, creek gravels and moved eastward against the canal lining. Collective flows from the underdrain system exceeded the capacity of the system and, when hydrostatic pressures became extensive, the lining ruptured. Following the cracking and displacement of the canal lining, filter sands from behind the lining piped out through the open cracks and the lining settled back into the resulting voids. Damage repair required removal and replacement of all lining and filter sand on the right side for over 1,400 feet and improvements in the underdrain system.

Modification of the underdrain through the damaged section included the following: (1) increase the collector pipe size to an 8-inch diameter; (2) replace the fine (Type B) filter sand with coarse (Type A) filter sand; and (3) excavate a longitudinal cutoff trench on both sides of the canal and backfill with impervious material to cut off the most permeable of the Quinto Creek gravels, thus preventing similar future flows against the lining.

At Garzas Creek, the underdrain pipes were in place, but neither the filter blanket nor canal paving had been constructed on the right side when ponding developed adjacent to the right side of the canal. This caused ground water inflow in the exposed right canal subgrade. The drain pipe was replaced with 8-inch-diameter pipe, coarse (Type A) filter sand was used in the blanket, and cutoff trenches were excavated and refilled with impervious material to approximate invert depth adjacent to both sides of the canal through the problem area.

In the construction of the checks and siphons and the Quinto and Romero Creeks overchutes, extensive dewatering systems usually were required. In some instances, cutoff or drainage trenches were sufficient; however, some type of well point system usually was needed.

When the first 4-foot by 4-foot overchute constructed lost more than double the amount of camber expected after form stripping, the camber was increased on the remaining smaller overchutes. Although structurally sound, the first overchute has a noticeable sag.

Initial Operations

In anticipation of seepage, both from and into the Aqueduct, water-level observation wells and piezometers were installed at strategic locations between Delta Pumping Plant and O'Neill Forebay. The observation wells furnished information on any changes in ground water levels in relation to pre- and post-construction periods. This information proved valuable in evaluating the performance of the lining of the Aqueduct and in observing effects on adjoining properties.

A system of levels with benchmarks was established on bridge overcrossings, principal drainage structures, and the aqueduct embankment itself at strategic locations to monitor any settlement which might occur. These measures, together with visual inspection, were part of the planned surveillance program during initial filling and operation of the Aqueduct.

Intake Channel to Delta Pumping Plant

The completed intake channel, before the plug was removed to connect it to Italian Slough, had accumulated about 500 acre-feet of ground water inflow. Unfortunately, this water was of very poor quality, the worst feature of which was that it contained 57 to 60 parts per million of boron. If this water were mixed with Delta water by removing the plug, the mixture would have a boron concentration at least 10 times that allowable for irrigation water.

The problem was resolved by pumping about three-fourths of the water into a holding evaporation pond before breaking the plug at Italian Slough. The pond was located within department-owned land adjacent to the intake channel and Italian Slough. Some reshaping had to be done within the pond to reduce overall pond size and to prevent inundation of the bases of several Bureau of Reclamation transmission line towers.

The ponded water evaporated in part and also was pumped back into Italian Slough at a rate of 200 gallons per minute. This rate provided adequate dilution by mixing with the tidal flows and with water then being withdrawn from the Delta by Delta Pumping Plant.

Delta Pumping Plant to Chrisman Road

This section of the Aqueduct was first filled in November 1967. Minor leakage was observed in numerous locations, particularly in the vicinity of Mountain House Road south of Bethany Forebay. The seepage decreased and stopped within a year, indicating a sealing of leaky joints and passageways with silt and clay particles. However, the flow at first was sufficient to require small amounts of corrective drainage for adjacent property owners.

Of the six monitoring wells, the water level rose moderately in two and then receded after several weeks. The rise in water level could have been a result of winter rains and seepage.

Monitoring of the structure benchmarks indicated no appreciable settlement. There was settlement of the compacted backfill beneath the overhanging transition walls of the check structures, a difficult area to compact. The settled areas were refilled with sand, which was compacted by vibration.

Just south of Bethany Forebay, the normal canal embankment on the left side cracked and failed. The embankment was reconstructed over a length of 300 feet. A pressure ridge or uplift at the toe of this slope from water which had seeped out of the canal was the cause of the failure. Impervious surface materials had prevented the escape of the seepage. Sand drains were placed in the base of the reconstructed embankment to assure seepage escape.

Farther south, between Altamont Pass Road and Highway 50 overcrossing, a pattern of cracks appeared on the left side of the canal embankment. After about three months, cracking and settlement ceased; there was minor canal leakage during the cracking. Settlement in the minimally stripped topsoil may have caused the embankment movement.

Chrisman Road to Orestimba Creek

These reaches of the Aqueduct (Chrisman Road to Del Puerto Canyon Road and Del Puerto Canyon Road to Orestimba Creek) were filled for the first time in November 1967 and experienced much the same conditions as for the reach to the north, with all settlement or cracking of minor nature. The embankment settlement occurred within the initial six weeks of water filling and was limited to high fill areas. Settlement of the foundation soils may have contributed to the minor settlement; however, it is more probable that the settlement occurred within the embankments themselves.

Orestimba Creek to O'Neill Forebay

This section contained the most prevalent high ground water conditions within the Division. Sumps constructed during establishment of the underdrain system were kept dry by pumping until after filling of the Aqueduct to prevent back-pressure on the lining. There were 16 monitoring wells established in the section. Only one well showed any seepage, and a toe drain at this location was sufficient to prevent any local property damage.

Settlement of embankments, structures, or refill areas around overhanging check transitions followed the pattern of other reaches in the Division. The most persistent evidence of seepage was adjacent to Orestimba and Garzas Creeks. Near Orestimba Creek, a saturated area appeared on the left side of the canal at the toe of a waste bank. After several weeks, the area dried up when silting sealed off the flow source.

At the north edge of Garzas Creek, two areas of ponds or springs developed. Tests of water samples indicated the major source for these springs was leakage from the Aqueduct. Divers found some open joints and leaking but, because of the murky water, observations were inconclusive. Experimental attempts to seal the openings with an asbestos-bentonite mixture, which was mixed at the surface and pumped to the openings through a hose held by the divers, were inconclusive. Later, a diver filled the openings with "Stop Leak", a commercial crack sealant. Latest observations indicate that flows and ponds adjacent to the canal were much reduced by this remedial work.

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CHAPTER V. SAN LUIS JOINT-USE FACILITIES

Introduction

The San Luis Joint-Use Facilities are the integrated works of the U.S. Bureau of Reclamation's Central Valley Project and State of California's State Water Project. They extend from the north end of O'Neill Forebay near Los Banos to Kettleman City, a distance of about 106 miles (Figure 135). The Bureau of Reclamation provides water to a service area of approximately 500,000 acres along the west side of the San Joaquin Valley. Under federal nomenclature, it is known as the San Luis Unit of the Central Valley Project. The State designates this reach of the California Aqueduct as the San Luis Division.

The San Luis Unit was federally authorized by Act of Congress, PL-488, which became law on June 3, 1960 (74 Stat. 156). State authorization, common to all features of the State Water Project, was by the Burns-Porter Act in 1960, as discussed in Volume I of this bulletin. Related references and reports covering the work discussed herein are listed at the end of this chapter. The Bureau of Reclamation has or will publish detailed technical records on the San Luis Unit. For this reason, this chapter purposely is kept brief.

The route for conveying project water to state service areas south of the San Luis Division is through the federal San Luis service area. Each project required development of the San Luis Dam site, west of Los Banos, for storage of surplus flows pumped from the Sacramento-San Joaquin Delta. Therefore, the optimum development for California and the United States was to integrate the storage, pumping, and conveyance facilities for coordinated operation. An agreement on December 30, 1961 achieved those objectives.

The agreement provided that the Bureau of Reclamation would design and construct the San Luis Joint-Use Facilities and cost sharing would be basically 55% state and 45% federal. The State assumed responsibility for operation and maintenance, and a later agreement provided for sharing that cost in accordance with the same formula. Lands and rights of way for San Luis Dam and Reservoir and O'Neill Dam and Forebay were obtained by the State. Titles to property secured by the State later were transferred to the United States.

The federal authorization act did not contain construction funds. State funds from the Water Resources Development Bond Act were advanced to the Bureau of Reclamation in 1961 for commencing design, and federal appropriation came later.

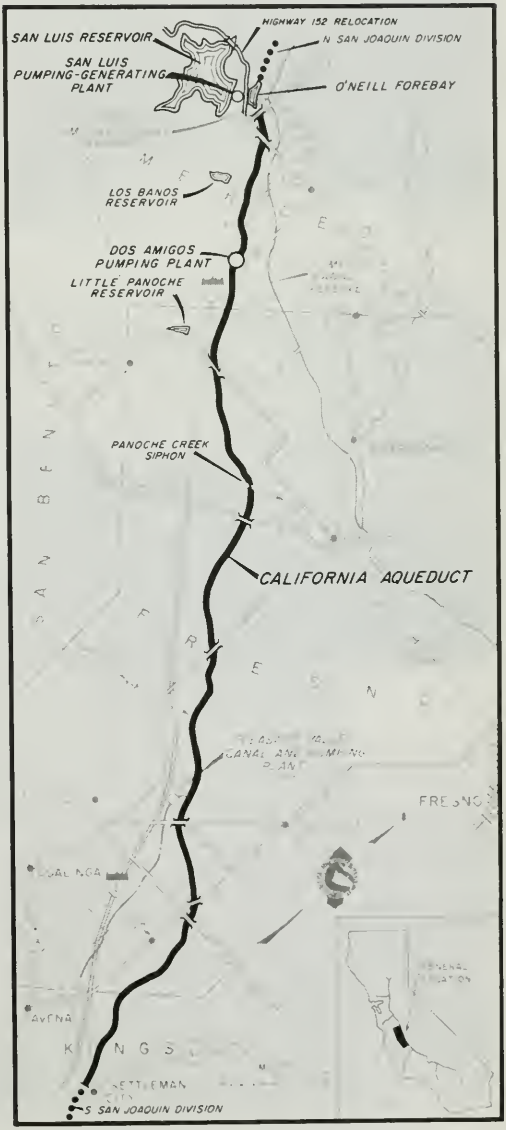

Figure 135. Location Map—San Luis Facilities

Figure 136. President John F. Kennedy and Governor Edmund G. Brown at Ground-Breaking Ceremonies

San Luis Unit ground-breaking ceremonies were held on August 18, 1962. The event was highlighted by the presence of President John F. Kennedy (Figure 136).

San Luis Canal, as constructed by the Bureau of Reclamation, consists of Reaches 1 through 5, which are now designated Reaches 3 through 7 of the California Aqueduct. They are not completely coincident reach by reach. The federal designation was for the purpose of orderly design and construction nomenclature which is used hereafter in this discussion. California Aqueduct designation by the Department of Water Resources at the beginning of Reach 1 is Milepost 70.64, which the Bureau of Reclamation designates Milepost 0.71 for San Luis Canal.

The capacity of San Luis Canal (Figure 137) varies from 13,100 cubic feet per second (cfs) in Reach 1 to 8,350 cfs in Reach 5. At the southern terminus at Kettleman City, Check No. 8 (California Aqueduct Check No. 21), the Canal can convey 8,100 cfs to the State during months when federal upstream demands are less than 5,000 cfs. During peak irrigation demand periods, however, it is possible to deliver only 7,000 cfs through Check No. 8. The peak federal summer irrigation demand is projected to be 6,000 cfs. The sum of all canal losses is assumed to be about 100 cfs.

Figure 137. San Luis Canal Prior to Filling

Location

San Luis Canal extends about 102.5 miles southeasterly from O'Neill Forebay to a point near Kettleman City (Figure 135). It substantially parallels Interstate Highway 5 located on the western side of the San Joaquin Valley at the eastern flank of the Coast Ranges. Most of the principal San Luis features depicted on Figure 135 were a joint undertaking by the United States and the State of California. The remainder of the principal facilities and all of the distribution and drainage systems were solely federal systems designed, built, and funded by the United States acting through the Bureau of Reclamation.

Figure 138. San Luis Canal

Figure 139. Typical Topography of West Side San Joaquin Valley

San Luis Canal water of the federal Central Valley Project serves the 500,000-acre San Luis federal service area, mostly for agricultural purposes and for some municipal and industrial uses (Figure 138). In addition, water of the State Water Project is transported through San Luis Canal directly to Kettleman City, the start of the Department's South San Joaquin Division of the California Aqueduct.

Federal service provides a firm supply of good quality water to existing and developing agricultural lands on the west side of the San Joaquin Valley. This service was intended to reduce the need to pump from deep aquifers, which was causing a ground water overdraft resulting in regional land subsidence.

Geology

San Luis Canal traverses the western side of the San Joaquin Valley along the eastern flank of the Diablo Range (Figure 139). The Diablo Range is formed by a broad anticlinal structure which has a central core composed of slightly metamorphosed sedimentary and igneous rocks of the Franciscan formation. The eastern flank of this large structure is formed by a thick sequence of Cretaceous-Age marine sandstone, shale, and conglomerate. These sedimentary beds, which are part of the Panoche formation, dip fairly uniformly eastward toward the center of the San Joaquin Valley. In some areas, the Panoche formation is in turn overlain by soft, Tertiary-Age, marine, sedimentary rocks also dipping to the east and forming low foothills along the western side of the Valley. Unconsolidated continental deposits of the Tertiary and Quaternary Ages also are present in the foothills of the Diablo Range.

The San Joaquin Valley is a structural trough filled with several thousand feet of alluvium derived from the Sierra Nevada Range to the east and the Diablo Range on the west. San Luis Canal traverses an area of piedmont deposits composed of detritus derived almost entirely from the Diablo Range. Alluvial fan material to a maximum depth of about 30 feet was encountered in the canal excavation.

The presence in some areas of high ground water tables, fat clays, and expansive clays created problems during design and construction. The soil structure was of great significance because of its potential for land subsidence.

A substantial portion of the canal alignment crosses areas subject to shallow subsidence in the upper 25 to 50 feet where the in-place soil density ranges from 80 to 100 pounds per cubic foot. Below 50 feet, the natural densities range from 100 to 110 pounds per cubic foot. In some areas, however, the low-density materials extend to depths of 100 feet.

When moisture is added through irrigation or canal seepage losses, these low-density materials undergo soil structure alteration and become more compacted. Thus, the land will settle by shallow subsidence by as much as 9 feet and possibly more (Figure 140).

Figure 140. Preconsolidation Ponds—Reoch 2

Compounding the problem of shallow subsidence is the phenomenon of deep subsidence. This is caused by extracting ground water from the deep aquifers at a rate in excess of their recharge capability.

Description

A statistical summary of San Luis Division conveyance facilities is presented in Table 9. Numerous petroleum and irrigation pipeline crossings have been omitted from the table.

San Luis Canal is a concrete-lined section with 2:1 side slopes and a bottom width varying from 110 feet in Reach 1 to 50 feet in Reach 5. The concrete lining is unreinforced and has a uniform thickness of 4½ inches. Contraction joints with a rubber filler and mastic cap are spaced at 15-foot intervals longitudinally and transversely. The Canal is operated as a closed system without wasteways or equalizing reservoirs. This system of operation, the "controlled volume concept", is more fully described in Chapter I of this volume and Volume V of this bulletin.

Some orientation details are shown on Figure 141, and the hydraulic gradeline and related data are shown on Figure 142.

Reach 1, an extension of O'Neill Forebay, is subject to water surface fluctuations between elevations 217 feet and 225 feet. Therefore, a complete system of lining protective drains, sand-filled finger drains, and flap-valve weeps was provided in this reach (Figure 143). This drainage system prevents excessive hydrostatic pressure from rupturing or displacing the lining. The canal inlet structure, which has four 32¼-foot-high by 25-foot-wide radial gates, is located at the beginning of Reach 1. These gates normally are in the fully open position. Reach 1 has a flat bottom grade at elevation 192.2 feet, with maximum water surface at elevation 225.0 feet. Reach 1 can be drained in case of an operational emergency by discharging into the culverts carrying Los Banos Creek flows under the Canal.

Reaches 2 through 5 have a constant bottom slope of .00004 (2.5 inches per mile). Nominal freeboard to the top of the concrete lining is 3 feet. A maximum 12 feet of freeboard was provided for future canal settlement because of the expected effects of land subsidence (Figure 142).

Deep subsidence originally was expected to be arrested as canal water supplanted pumped ground water. This change in water source was not developed fully in the time originally anticipated because of a delay in funding of distribution systems, so regional deep subsidence continues to some degree. In Reach 3, 17,000 lineal feet of canal was provided with 12 feet of freeboard where the canal invert also was set 2 feet above the nominal grade. It was estimated that postconstruction settlement would bring the bottom grade down to or below nominal invert grade.

In some areas along the Canal, there are high, perched, ground water tables. In other areas, there are deposits of fat clays that have poor drainage characteristics. In additional areas, as for the entire alignment of Reach 1, lining protective drains and finger drains with flap-valve outlets were provided.

Canal cross sections and details of the concrete lining, protective drains, contraction joints, embankments, and hydraulic properties for each of the reaches are shown on Figures 143 through 147.

Cross drainage is handled by passing it under the Canal in culverts; admitting it into the Canal at drain inlets; providing a double-barreled, 28-foot, concrete siphon at Panoche Creek (Figure 148); and ponding it against the canal bank. Ponded water, which is not taken into the Canal or passed under it, dissipates by evaporation and percolation.

Los Banos and Little Panoche Detention Dams and Reservoirs protect the Canal from floodwaters by detaining the water and gradually releasing it. These dams are described in Volume III of this bulletin.

Standby generators are housed at each of the eight check structures to operate the gates during primary power failures. The enclosures also protect the controls which operate the check gates and the sump pumps which evacuate the check-structure drains.

TABLE 9. Statistical Summary of San Luis Division

CANAL	
Type	Concrete-lined—trapezoidal—checked
Dimensions	Lined depth, varies from 36.8 to 25.1 feet; bottom width, varies from 110 to 50 feet; side slopes, 2:1; length, 102.5 miles
Capacity	Variable in steps from 13,100 cubic feet per second at inlet to 11,800, 9350, and 8350 at Check No. 21
Freeboard	Varies from 3 to 10 feet lined and a minimum of 2.5 feet earth berm above lining—depth of lining dependent upon anticipated subsidence
Lining	4½-inch unreinforced concrete—sealed longitudinal and transverse contraction joints on a maximum of 15-foot centers
Bridges	49 vehicular—2 railroad—1 pedestrian
Check Structures	4 four-radial-gate structures
Cross-Drainage Structures	5 culverts—51 drain inlets
Canal Drain	One at Los Banos Creek
SIPHON	One at Little Panoche Creek
OPERATIONS	Manual on-site control or remote control from area control center, San Luis Field Division

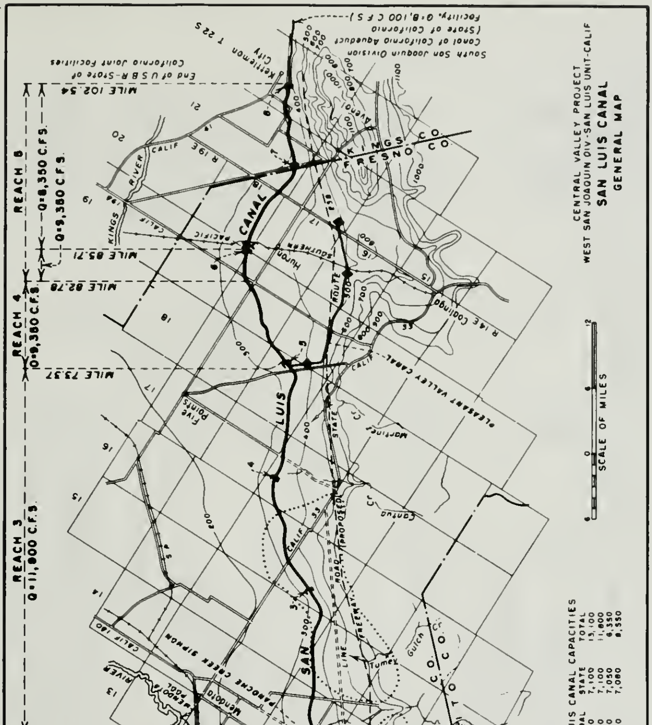

Figure 141. General Map—San Luis Canal

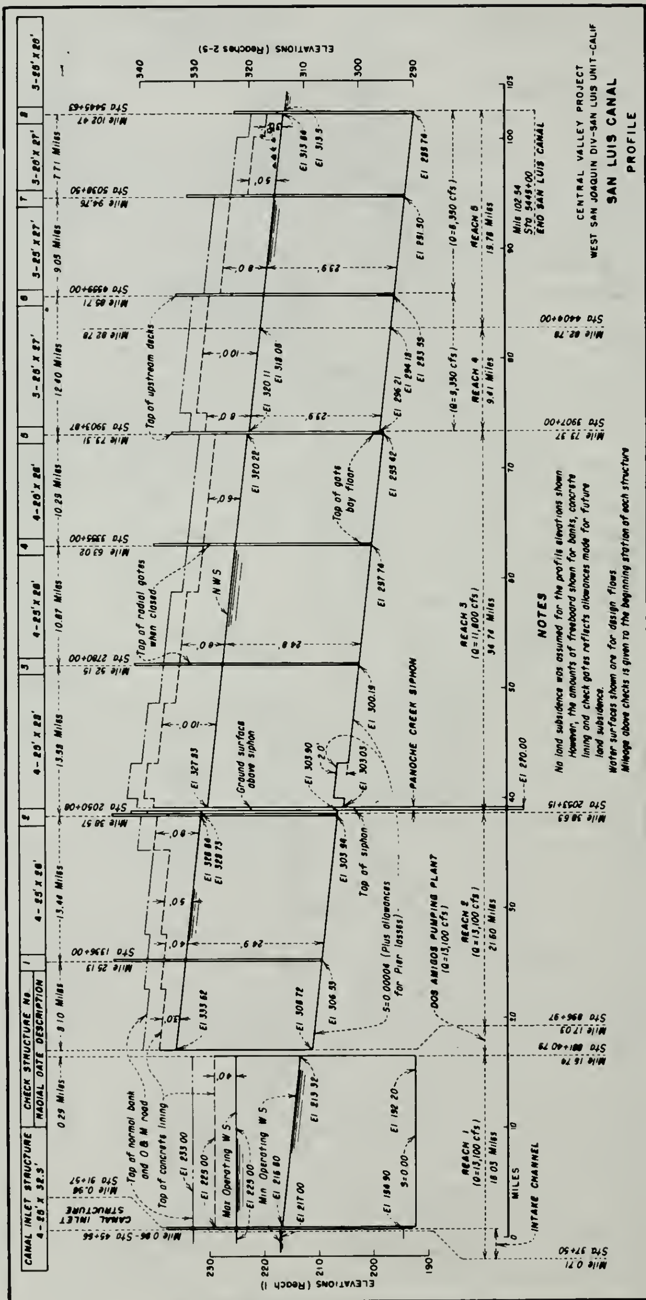

Figure 142. San Luis Canal Profile

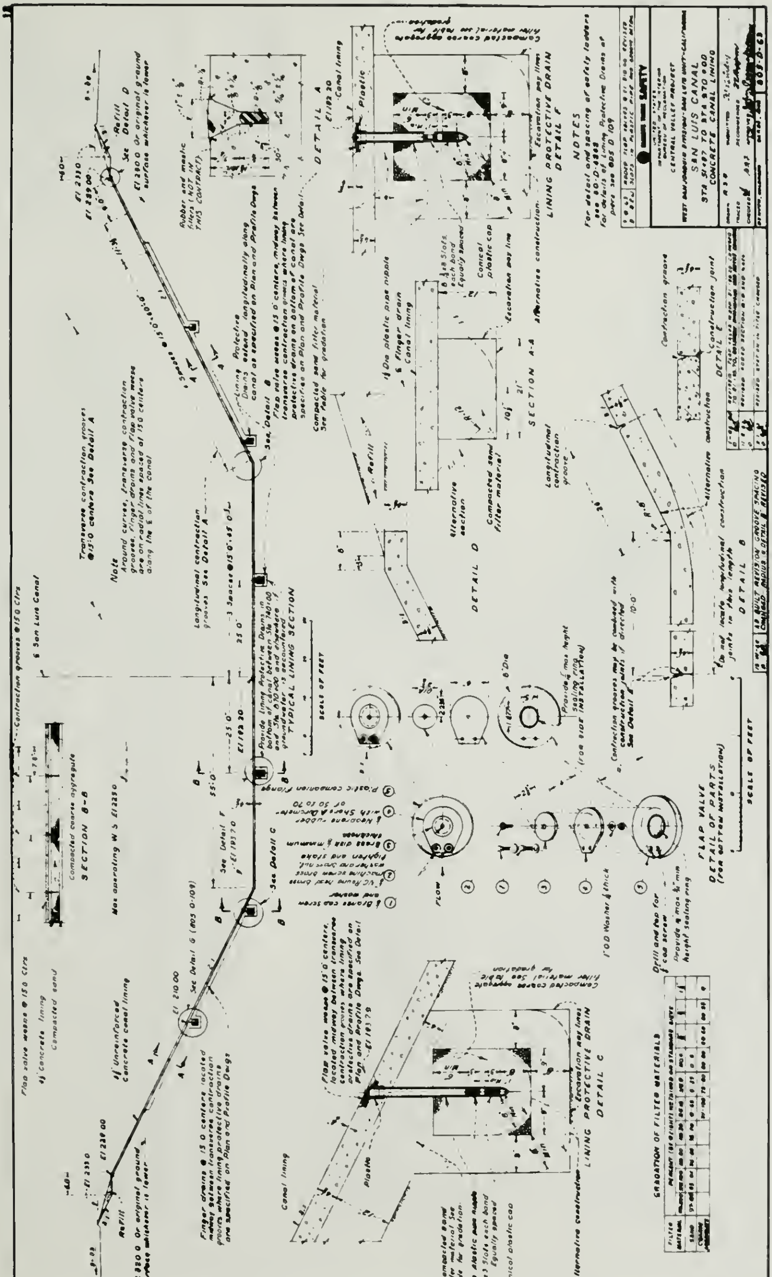

Figure 143. Concrete Canal Lining—San Luis Canal

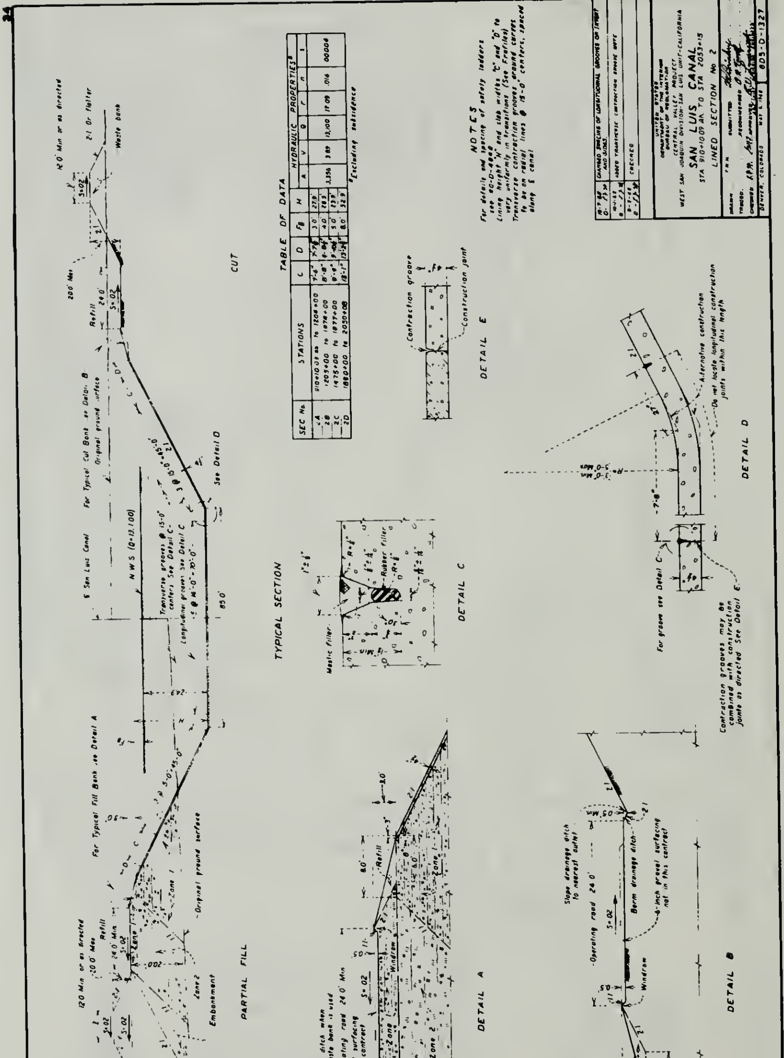

Figure 144. Lined Section No. 2—San Luis Canal

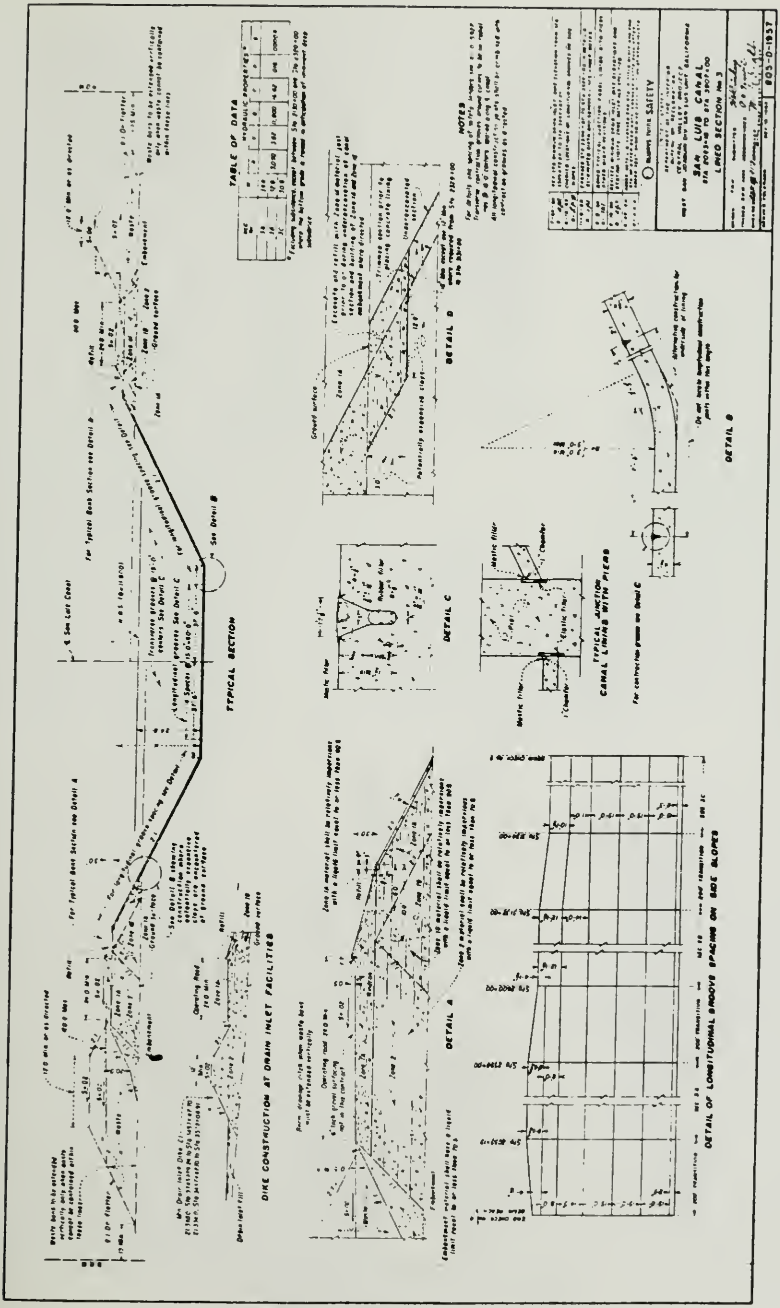

Figure 145. Lined Section No. 3—San Luis Canal

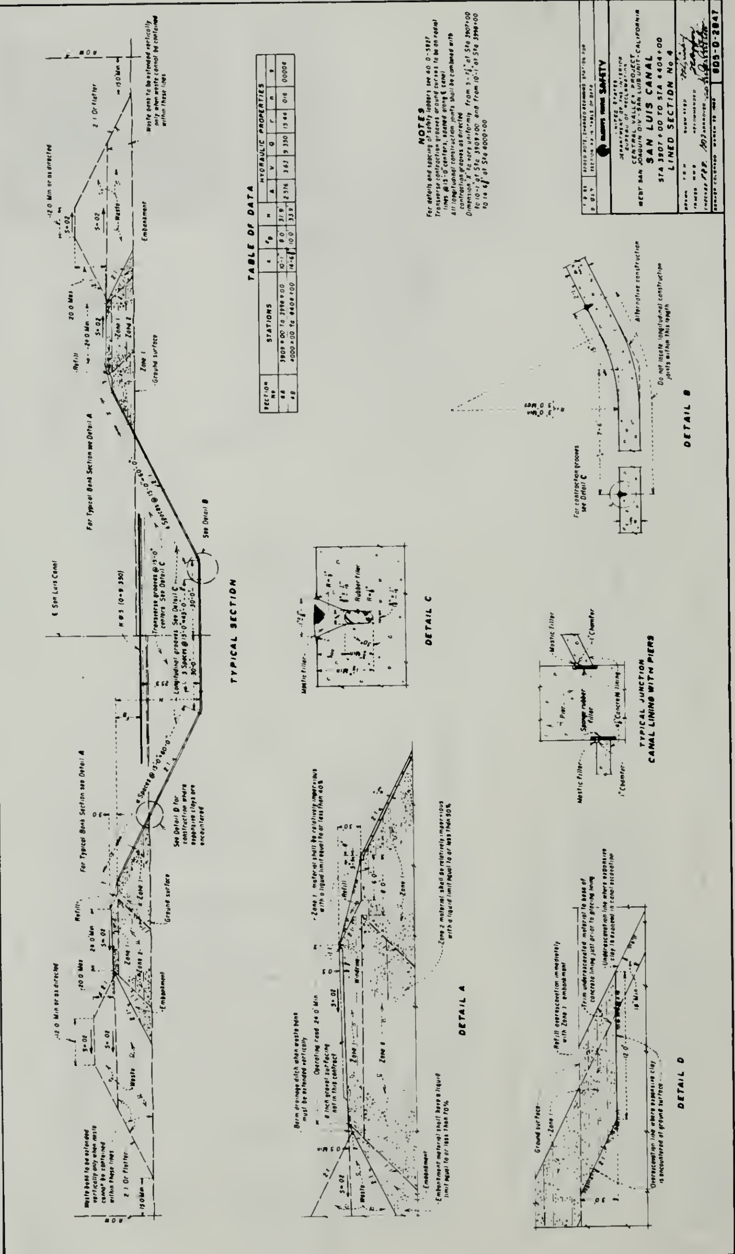

Figure 146. Lined Section No. 4—San Luis Canal

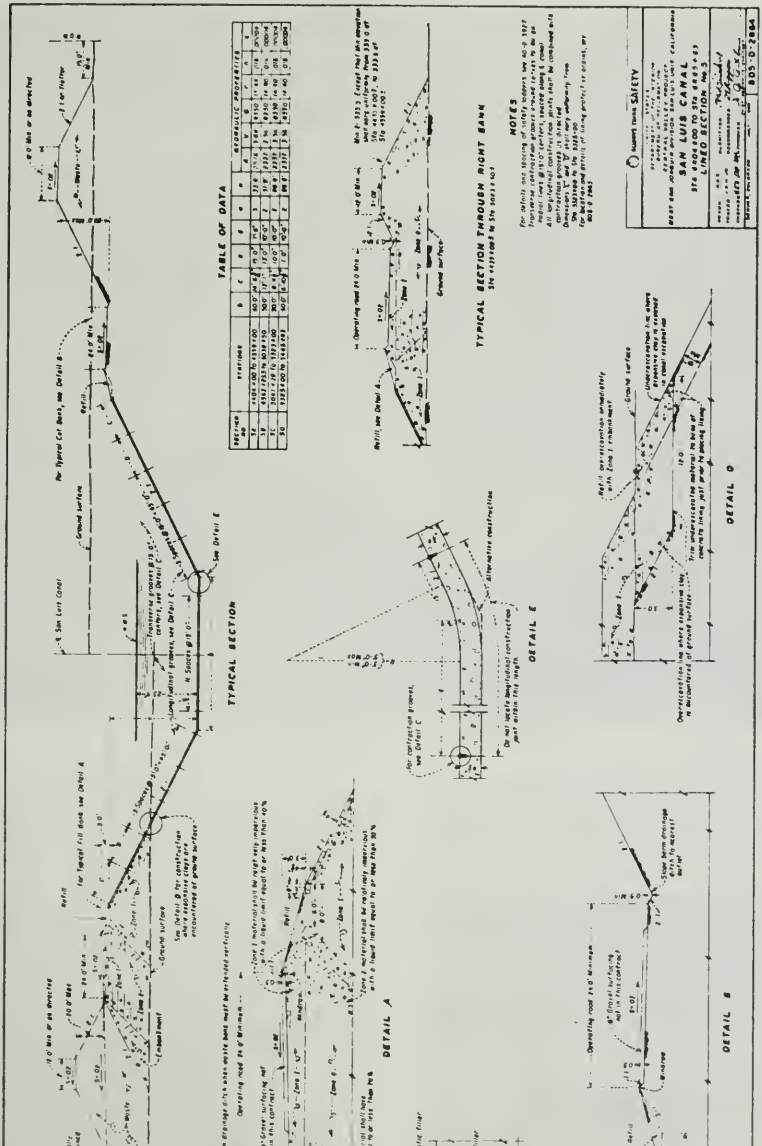

Figure 147. Lined Section No. 5—San Luis Canal

Figure 148. Siphon Construction—San Luis Canal

The entire canal is remotely controlled from the area control center located in the San Luis Pumping-Generating Plant. Remote control and monitoring equipment was provided by the State.

Automation enables simultaneous operation of all check gates, which is essential to achieve changes in canal flow to match variations in pump flow and thus obtain the benefits of off-peak pumping. This is described fully in Volume V of this bulletin.

No provisions are made for direct draining of the Canal in Reaches 2 through 5. Pumping over the next downstream check will be required when dewatering any portion of these reaches. The nearest canal drain is 46 miles beyond the end of San Luis Canal in the South San Joaquin Division. This drain, located northwest of the town of Buttonwillow, discharges into the Kern River. Ordinarily, San Luis Canal will never be dewatered except during an extreme emergency. Emergency dewatering of canal reaches downstream of Dos Amigos Pumping Plant may be accomplished by use of eight portable pumps. The water will be pumped over the radial gates of the check structure at either end of the reach and into the adjacent reach.

Construction

The magnitude of construction is illustrated by the volumes of materials handled (Table 10).

Subsidence areas of the canal alignment were preconsolidated to minimize settlement and consequent distress in the canal lining and related structures.

Where in-place densities were less than 100 pounds

TABLE 10. Major Volumes of Materials Handled
(Data from Invitations for Bids)
San Luis Canal

Reach	Canal Excavation (cubic yards)	Concrete Construction	
		Canal Lining (cubic yards)	Other (cubic yards)
1	15,500,000	315,100	40,750
2	14,465,000	345,000	49,280
3	18,500,000	560,000	51,300
4	4,550,000	146,000	4,990
5	9,670,000	273,600	28,610
Total	62,685,000	1,639,700	174,930

per cubic foot, foundation treatment by preconsolidation with ponded water was necessary. Work was awarded under separate contract prior to contracting for construction of the Canal.

Preconsolidation was accomplished by constructing 103 holding ponds averaging 770 feet long by 360 feet wide. In some areas, infiltration wells were drilled to a depth of 125 feet. The ponds were filled with water and replenished continuously until a condition of equilibrium was reached such that future consolidation from agricultural water application to adjacent farm lands would be of small magnitude.

The infiltration wells were augered and gravel-filled to admit the water at depth. They were 100 feet apart in two rows at 43 feet on each side of the canal centerline. Later, the gravel was removed to a depth of 3 feet below the canal lining and backfilled with compacted embankment material.

Rough excavation in areas of expansive clay extended to within 18 inches of final grade to prevent loss of natural moisture. Because these clays expand only after drying, their natural moisture content was maintained by sprinkling.

Construction of this reach of canal followed the same procedure as the other reaches (Figures 149 and 150). However, some new equipment was tried to help in the earth-moving task (Figures 151 and 152).

Initial Operations

Water-level observation wells were installed at various locations within the San Luis Canal right of way to monitor changes in ground water levels relative to pre- and post-construction times. This information proved helpful in evaluating performance of the lining and in observing effects of the Canal on adjoining properties.

A system of levels with benchmarks to monitor settlement which may occur was established on bridges crossing the Canal, check structures, various other associated structures, and the canal lining in certain locations. These measures, coupled with visual inspection, constituted the planned surveillance during initial filling and operation of the Canal.

Reach 1

Seepage from the Canal to adjacent farm lands resulted in the installation of an interceptor drain along the left right-of-way line. This 6,300-foot-long drain extends from about one-half mile upstream of the Interstate 5 bridge crossing to about one-half mile downstream of the bridge. An 8-inch-diameter open-joint drain was constructed in 1968 to intercept seepage from the Canal that was moving into adjacent farm lands. Water from the drain is collected in a series of four sumps and pumped back into the Canal.

Reach 2

Differential settlement attributed to localized shallow subsidence occurred at various bridge crossings. In order to maintain relatively level bridge decks, it was necessary to raise portions of various bridge decks. This work, using hydraulic jacks and concrete block shims, was done on the following bridges:

Bridge	*Part Raised*	*Amount (inches)*	*Date*
Eagle Field Road	Left Abutment	13	7/68
"	Right Abutment	18	7/68
"	Both Abutments	3	4/69
"	Left Abutment	3	1/71
Nees Avenue	Left Pier	3	12/70
Shields Avenue	Left Abutment	3	1/71

Experience gained from 1969 flooding resulted in the acquisition of two 20-cubic-foot-per-second portable pumps and the construction of 30 pump sumps along the right side of the Canal. Two sumps were located in Reach 2. Pumping storm runoff ponded against the canal dike into the Canal as quickly as possible minimizes damages to adjacent farm lands.

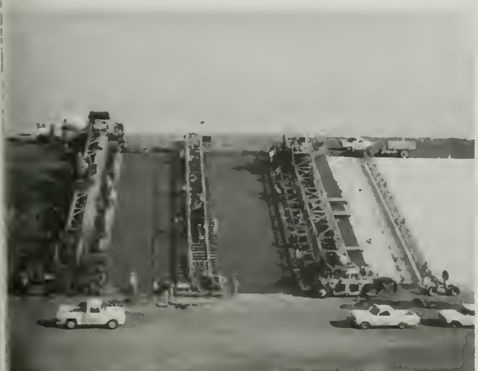

Figure 149. Side Slope Paving Train Operating in Sequence

Figure 150. Canal Invert Lining


Figure 151. Triple Earth-Moving Unit

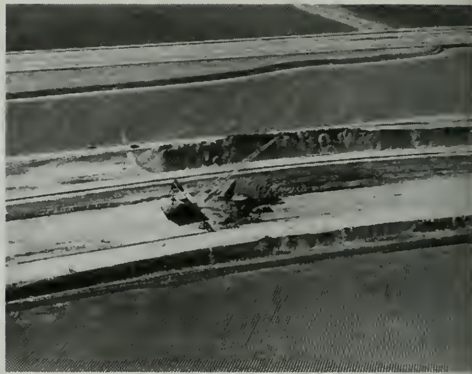
Figure 152. Walking Dragline Excavating Canal Prism

Reach 3

Differential settlement of Manning Avenue bridge required the following: (1) left abutment raised 4½ inches and right abutment 16 inches in January 1969, (2) right abutment raised 3 inches in September 1969, and (3) right abutment raised 3 inches in January 1971. Twenty-three portable pump sumps were constructed in this reach to facilitate pumping of ponded water into the Canal.

During January 1970, 95,000 cubic yards of silt was dredged from the Canal between Check No. 4 and Mt. Whitney Avenue bridge. The silt was deposited by Cantua Creek flowing through an uncontrolled drain inlet into the Canal during the 1969 floods. This drain inlet has been sealed and replaced with a weir-board controlled flume of equal capacity.

A dike extending westerly from the Canal and located upstream of Fresno-Coalinga Road was constructed in 1971. This dike precludes storm runoff, which passes through a large feed lot, from reaching a drain inlet or pump sump from where it could enter the Canal.

Settlement attributed to deep subsidence necessitated considerable modification work in this reach, namely:

1. Lining was raised 2 feet for a distance of approximately 4,000 feet upstream of Check No. 4 in 1969.

2. Lining was raised 3 feet between Clarkson Avenue and Parkhurst Avenue, except for that portion which was raised an additional 2 feet in 1969. Gate sills of Check No. 4 were raised 3 feet in 1971. In 1973, the turnouts in this section of canal were modified to accommodate settlement.

3. San Mateo Avenue, Cerini Avenue, and Mt. Whitney Avenue bridges were raised 4½ to 5½ feet in 1973.

Reach 4

Five portable pump sumps were constructed in this reach to facilitate pumping of ponded water into the Canal.

Reach 5

Settlement attributed to deep subsidence and silt deposited in the Arroyo Pasajero impounding area during the 1969 floods reduced floodwater handling capabilities. Therefore, the Canal protective dike and the east-west training dike were raised 4 feet between Dorris Avenue and Gale Avenue in 1972. Also, in 1969, Lassen Avenue bridge at the east-west training dike was severely undercut. Invert and slope protection by means of sheet piling and sacked concrete were provided at this bridge in 1970.

Twelve gated drain inlets were installed at the lower end of Reach 5 to prevent overtopping of the low canal embankment by flood runoff. The gates are closed in the off-season to stop field runoff irrigation waters from entering the Canal.

The east-west training dike has been subjected to erosion a number of times even with light-to-moderate flows to the impounding area. To reduce these erosion problems, the upstream side of this dike was furnished protection in the form of rock-filled gabions for 1,100 feet west of Lassen Avenue and 2,200 feet east of Lassen Avenue in 1974.

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CHAPTER VI. SOUTH SAN JOAQUIN DIVISION

Introduction

Role in the State Water Project

The South San Joaquin Division is the third and last division of the California Aqueduct to originate and end in the San Joaquin Valley. This division begins at Kettleman City at the end of the San Luis Joint-Use Facilities and terminates at A. D. Edmonston Pumping Plant (Figure 153). The Coastal Branch forks from this section of the Aqueduct a few miles south of Kettleman City. The South San Joaquin Division is distinctive in that it serves the largest number of the State Water Project's agricultural water users. These facilities were designed and constructed during the period 1960 to 1971.

Figure 153. Location Map—South San Joaquin Division

Hydraulic Function

Flow in the Aqueduct (Figure 154) is conveyed in a concrete-lined canal section with the dimensions of the section reducing as successive water deliveries are made from the various turnouts. Capacity changes from 8,100 cubic feet per second (cfs) just below Kettleman City to 4,400 cfs at A. D. Edmonston Pumping Plant. Three pumping plants in the southern one-third of the Division provide a total lift of 956 feet as the Aqueduct rises through the foothills to the base of the Tehachapi Mountains. Slope of the Aqueduct is 0.00004 feet per foot from Kettleman City to the first (Buena Vista) pumping plant, changing to 0.000045 feet per foot for the remainder of the Division. There are a total of 15 check structures, 12 siphon structures, 2 wasteways, and over 40 water delivery turnouts within the Division.

Geography, Topography, and Climate

The Division lies wholly within Kings and Kern Counties. For approximately the first half of its 120 miles, the Aqueduct, lying mostly in the flat almost desertlike valley floor, skirts the eastern edge of the Coast Range foothills. From Tupman Road to Buena Vista Pumping Plant, the canal follows the western edge of the normally dry bed of Buena Vista Lake. This lakebed is a terminal sink for floodflows of the Kern River which drains the Sierra Nevadas to the east.

The Aqueduct in the southerly half of this division rises through elevated extensions of the bordering Coast Ranges and cuts through the foothills of the Tehachapi Mountains at Wind Gap to A. D. Edmonston Pumping Plant. This division is located in one of the least populated areas of the State. The southern portion is agriculturally oriented with large farms previously dependent on ground water pumping for a water supply.

Interstate Highway 5 parallels the Aqueduct, and a

Figure 154. California Aqueduct—Looking South From Dudley Ridge Turnout Near Kettleman City

drive through the Division now dramatically illustrates the change taking place with the availability of project water. Miles of orchards and lush crops rapidly are replacing the previously bleak landscape. Annual precipitation is less than 12 inches in this area, nearly all of which accumulates from a few high-intensity short-duration storms during the winter months. The summers are long and hot with daily temperatures frequently in the upper 90- or lower 100-degree range. During the winters, freezing nighttime temperatures are not uncommon.

Features

The Division begins at Check No. 21 (Milepost 172.40) near Kettleman City. The design capacity at this check is 8,100 cfs. The bottom width decreases from 50 to 32 feet and the side slopes are 2:1.

At Milepost 236.60, the side slopes of the Aqueduct change to $2\frac{1}{2}$:1 and remain so for a distance of 13 miles to the Buena Vista Pumping Plant forebay. Downstream of Buena Vista Pumping Plant, the side slopes are 2:1, the bottom width is 24 feet, the grade is 0.000045, and the design capacity is 5,050 cfs, at a depth of 22.9 feet. Eleven miles south of Buena Vista Pumping Plant, the canal capacity is reduced to 4,900 cfs and remains unchanged to Wind Gap Pumping Plant, 29 miles downstream from Buena Vista Pumping Plant.

At Wind Gap Pumping Plant, the design capacity is reduced to 4,400 cfs and remains unchanged to the terminus of the Division. There are no regulating or storage reservoirs in the South San Joaquin Division.

Twelve miles south of Kettleman City, the Coastal Branch forks from the main aqueduct. The Coastal Branch is discussed in the next chapter of this volume.

For design and construction purposes, the South San Joaquin Division conveyance facilities were divided into six sections: Kettleman City to Avenal Gap, Avenal Gap to 7th Standard Road, 7th Standard Road to Tupman Road, Tupman Road to Buena Vista Pumping Plant intake channel, Buena Vista Pumping Plant to Wheeler Ridge Pumping Plant, and Wheeler Ridge Pumping Plant to A. D. Edmonston Pumping Plant. A statistical summary of South San Joaquin Division conveyance facilities is presented in Table 11.

TABLE 11. Statistical Summary of South San Joaquin Division

CANAL	
Type	Concrete-lined—trapezoidal—checked
Dimensions	Lined depth, varies from 26.31 to 21.00 feet; bottom width, varies from 32 and 24 feet; side slopes, 2:1 and 2½:1; length, 121 miles
Capacity	Variable in steps from 8,100 cubic feet per second at head check to 7300, 6350, 5950, 5350, 5050, 4900, and 4400 at intake to A. D. Edmonston Pumping Plant
Freeboard	2.5 to 8.0 feet lined and a minimum of 2.5 feet of earth berm above lining—depth of lining dependent upon anticipated subsidence
Lining	4-inch unreinforced concrete—sealed longitudinal and transverse contraction joints on a maximum of 12½-foot centers
Bridges	54 vehicular—1 railroad
Check Structures	15 four-radial-gate structures—11 three-radial-gate structures
Cross-Drainage Structures	26 culverts—80 overchutes
Canal Drains	2, one at Kern River and one at Pastoria Creek
Spill Basin	One located 7,000 feet upstream from Buena Vista Pumping Plant
SIPHONS	12, located at Avenal Gap channel; Temblor, Sandy, Santiago, Los Lobos, San Emigdio, Old River, Pleitito, Salt, Grapevine, and Pastoria Creeks; and Sunset Railroad
OPERATIONS	Manual on-site control or remote control from area control center (Kettleman City to Buena Vista Pumping Plant intake channel, San Luis Field Division; Buena Vista Pumping Plant to A. D. Edmonston Pumping Plant, San Joaquin Field Division)

Geology and Soils

Regional Geology

The California Aqueduct extends along the west side of the San Joaquin Valley and then turns east at the south end of the Valley to the foot of the Tehachapi Mountains. The Valley is a broad, northwest-trending, structural trough which is bordered on the east by the Sierra Nevada, on the west by the Diablo and Temblor Ranges, and on the south by the San Emigdio and Tehachapi Mountains. A great thickness of sedimentary rocks, mostly marine in origin, have been deposited from ancestral seas that once filled the San Joaquin Valley but includes some nonmarine sedimentary deposits, particularly the younger fluvial and alluvial deposits that cover the valley floor and margins. These soft, younger, nonmarine deposits which range from Plio-Pleistocene to Holocene in age are the erosional detritus from bordering highlands and form large coalescing alluvial fans and broad alluvial plains. Nearly all of the Aqueduct in the South San Joaquin Division is underlain by these younger alluvial and fluvial deposits.

About 500,000 years ago during the mid-Pleistocene epoch, there was a small orogeny or mountain-building period in the area. During this period, sedimentary deposits around the margin of the Valley were folded into low hills which protrude through the alluvial floor of the Valley. Kettleman Hills, Lost Hills, Elk Hills, and Buena Vista Hills resulted from this last orogeny.

Special Geologic Considerations

During the planning stage, it was recognized that a water conveyance system would encounter geologic-related engineering problems in the South San Joaquin Division that were unusual in most other aqueduct projects. The major geologic problems were seismicity, shallow subsidence, and deep subsidence.

The southern end of the San Joaquin Valley, because of its tectonic relationship to the active mountain-building forces in the Coast and Transverse Ranges, has been shaken by many earthquakes. Six major faults and numerous minor faults occur within 30 miles of the aqueduct alignment. Three of the major faults, the San Andreas, White Wolf, and Santa Ynez, were responsible for destructive earthquakes within historic time. The other three major faults, the Garlock, San Gabriel, and Big Pine, dominant features in the mountain ranges to the south, have not been sources of damaging earthquakes within historic times but are viewed as being capable of generating large earthquakes. In short, with the considerable seismic activity in the southern portion of the South San Joaquin Division, as well as in the Tehachapi crossing and beyond, allowances were made in design for potential disruption of the Aqueduct by seismic disturbances.

Subsidence of land surface in the western and southern portions of the San Joaquin Valley has been recognized for many years. This subsidence is attributed to two causes: (1) a deep subsidence which results from the withdrawal of ground water and the concurrent compaction of the aquifer; and (2) shallow subsidence which results from the collapse of low-density open-structure soils when saturated, a phenomenon also known as hydrocompaction.

An area of deep susidence (Figure 155), with the center of maximum subsidence near U.S. Highway 99, is approximately 15 miles south of Bakersfield. The subsidence extends from Arvin to Wheeler Ridge and westward to Maricopa. Data obtained by the U.S. Coast and Geodetic Survey and mapping by the U.S. Geological Survey indicate most of the Aqueduct across the southern end of the Valley is within an area that has subsided 1 foot. Northwest of Wheeler Ridge Pumping Plant, the canal is near the end of an area that has subsided 4 feet. To prevent this type of broad gentle settlement from impairing the delivery of water through canals, additional freeboard was added to canal embankments to compensate for anticipated settlement.

Extensive areas of shallow subsidence (Figure 156) occur along the west and south side of the San Joaquin Valley and in El Rincon Valley south of Wheeler Ridge. Soils subject to shallow subsidence originally were deposited on the alluvial fans by debris flows. Debris flows are slurries of unsorted clay, silt, sand, gravel, large boulders, and plant debris that flow from the mountains during brief but intense rainstorms. The fluid masses of debris race down canyons onto alluvial fans where they spread out within a short distance. The debris flow dries rapidly on the alluvial fans leaving a soil structure with a large amount of voids. When saturated, such soils weaken and the voids collapse, reducing soil volume and causing settlement. After much study, most of the effects of shallow subsidence were solved by applying water and inducing subsidence prior to construction of the canal.

Hydrocompactive Soils

The problem of shallow subsidence when water is applied to some of the soils in the San Joaquin Valley is discussed briefly in Chapter I of this volume. It had been recognized for decades that these hydrocompactive soils existed (Figure 156) and would create a challenging engineering problem to the southward conveyance of water. However, it was not until the Department of Water Resources and the U.S. Bureau of Reclamation became interested in the construction of the California Aqueduct and the Division of Highways (now the Department of Transportation) in building the Westside Freeway (Interstate 5) that sufficient interest, manpower, and funds became available for a full-scale investigation of the problem.

In May 1954, a joint conference held in Washington, D.C. established a cooperative program to study subsidence. This conference led directly, in December 1954, to the formation of the Interagency Committee on Land Subsidence in the San Joaquin Valley. The

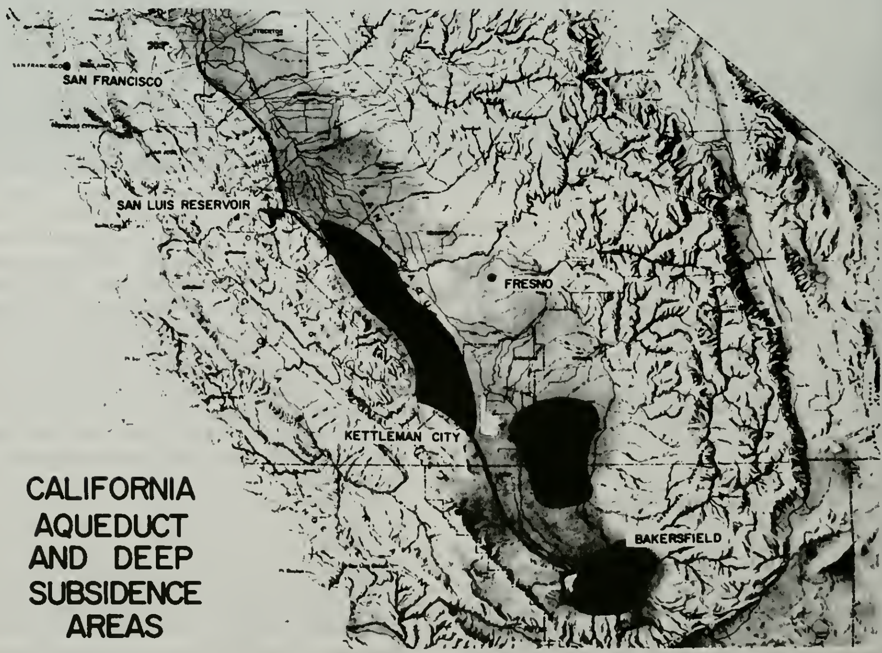

Figure 155. Areas of Deep Subsidence

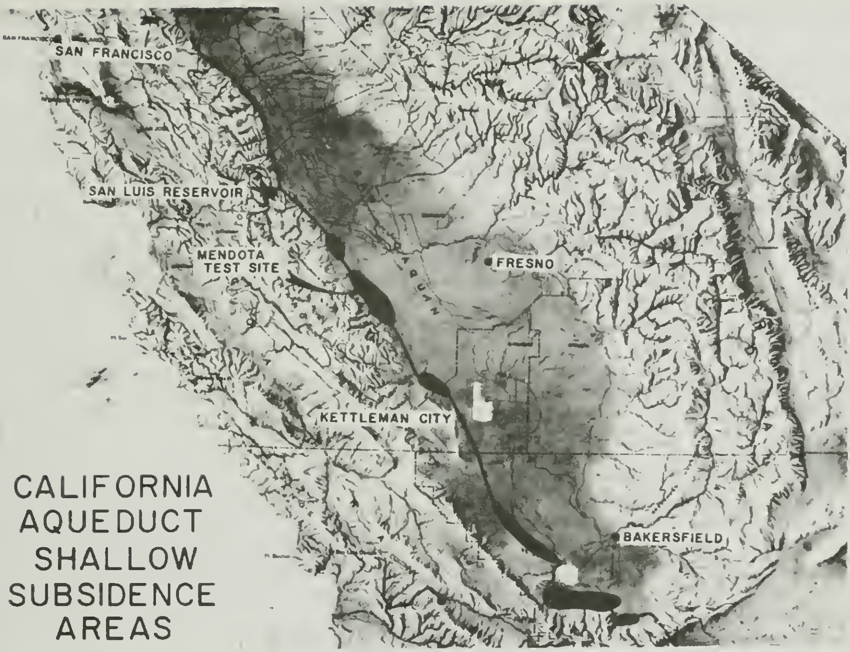

Figure 156. Areas of Shallow Subsidence

Committee was composed of federal representatives from the Bureau of Reclamation, Corps of Engineers, Geological Survey, Coast and Geodetic Survey, and the Soil Conservation Service and state representatives from the Department of Water Resources, the Division of Highways, and the University of California at Davis. The Committee was divided into three groups, each with a specific charge, to investigate (1) vertical control and topographic mapping, (2) shallow subsidence, and (3) deep subsidence.

A proposed program of investigation was prepared in 1955 by the Interagency Committee and, in 1958, a progress report on land subsidence investigations in the San Joaquin Valley was published.

One result of the interagency cooperation was an intensive study of land subsidence in the San Joaquin Valley by the U.S. Geological Survey, in financial cooperation with the Department of Water Resources. The Geological Survey reported some results of this investigation in 1972.

In 1957, the Department realized that forthcoming design and construction schedules for project facilities would require an additional and accelerated effort. Accordingly, an expanded program on shallow subsidence was initiated. This expanded program initially included the area of the San Luis Division; however, following the San Luis agreement in 1961, whereby the Bureau of Reclamation would design and construct the San Luis facilities (see Volume I of this bulletin), all information on subsidence which the Department had developed on that area was transferred to the Bureau of Reclamation.

The Department's expanded program was divided into two parts: a test site study and a route study. A 240-acre test site was selected about 15 miles south of Mendota to develop an economical technique for compaction of soils in subsidence areas and to collect data to provide a basis for route selection (Figure 157). The alignment phase was to delimit the areas of shallow subsidence, determine the rates and magnitudes of subsidence to be expected in those areas, and accomplish adequate treatment prior to construction. Data was obtained primarily from an extensive subsurface exploration program and test site investigations.

The alignment program consisted of exploratory drilling and sampling at selected locations approximately 5 miles apart along the anticipated alignment. Holes were drilled to various depths depending upon the characteristics of the samples obtained. Exploration was accomplished by utilizing compressed air as the drilling fluid and applying specialized sampling techniques. Soil samples thus

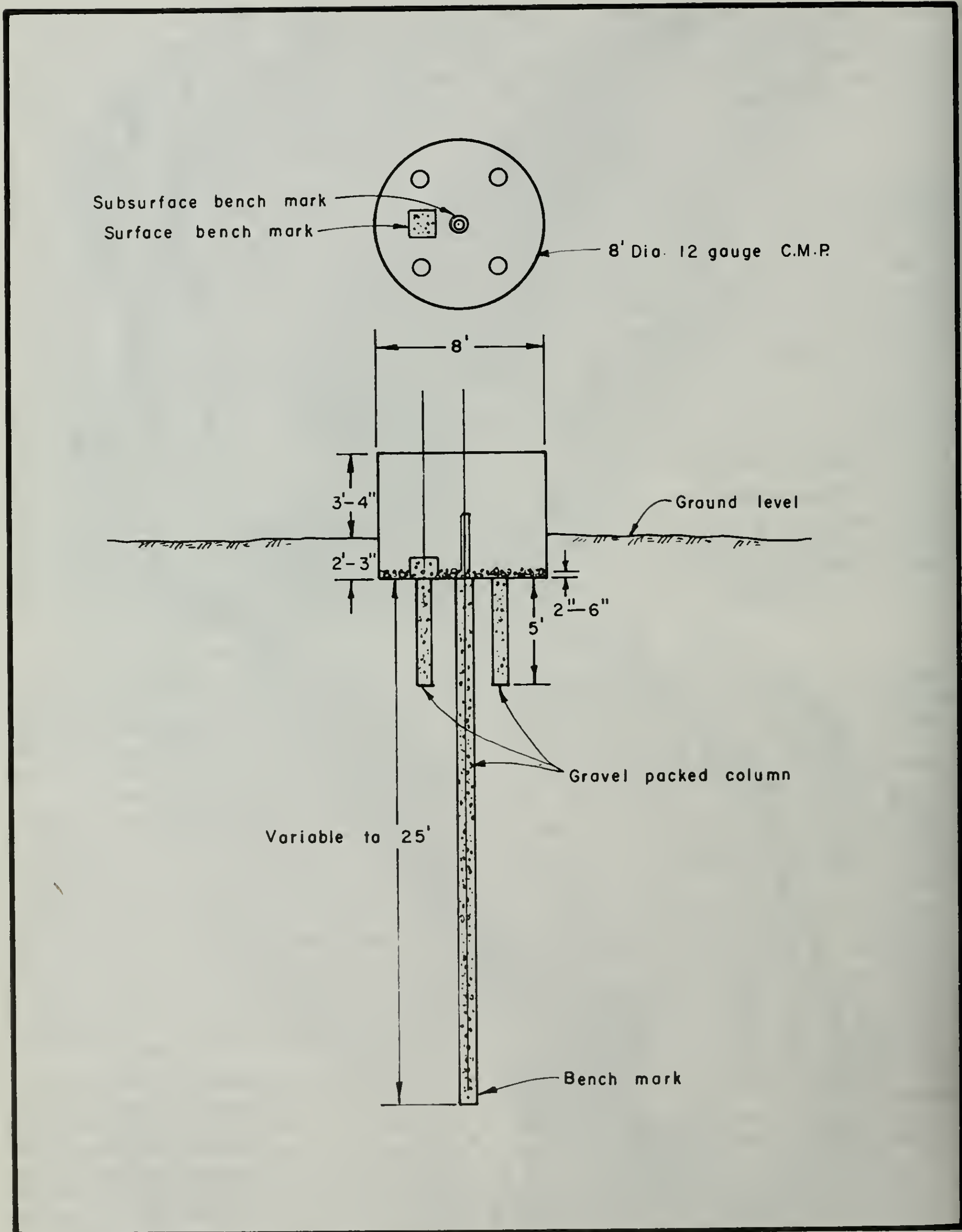

Figure 157. Typical Alignment Test Plot

provided were subjected to extensive laboratory testing.

Small test plots were installed at the sampled sites. A typical test plot consisted of embedding an 8-foot-diameter corrugated-metal pipe 2 to 3 feet into the soil. Four shallow (to 5 feet) and one deep (to 25 feet) gravel-packed wells were developed inside the perimeter of the pipe to increase the infiltration rate of the applied water (Figure 158). Usually, subsidence or swelling of the soil was observed a few hours after initial water application. After a few days, circumferential cracking occurred a few feet outside the pipe. As subsidence progressed, these cracks gradually widened to a few inches, and the soil block and tank settled. This continued action resulted in a typical, concentric, stair-stepped cavity.

The amount of subsidence along the alignment varied from 2 to 11 feet. The areas delineated which required special treatment prior to construction are shown on Figure 156.

At the Mendota test site, a study was made to determine the most effective method of water application, the optimum length of time required for water application, and the total required water. Four large and several small test plots were developed and operated. The first large plot was an unlined canal section. This section was a 200-foot by 400-foot rectangle with the depth of water kept at 1 foot. Water was applied for 484 days with an average settlement of 13.5 feet.

The second large plot was developed to evaluate the feasibility of utilizing shear cracks as a means of applying water at depth. A ditch 12 feet wide by 220 feet long was used and, as subsidence and cracking developed, the water was held at a constant elevation which resulted in the cracks becoming inundated. Water was applied for 346 days with an average subsidence of 11.5 feet.

The third large plot, also a 200-foot by 400-foot rectangle, was used to determine if the subsidence process could be accelerated by injecting water at depth through gravel-packed infiltration wells in conjunction with normal ponding. The plot was operated for 210 days with an average subsidence of 8.7 feet.

The last large plot was similar to the previous one except that the gravel-packed holes were spaced more closely. This plot was operated for 305 days with an average subsidence of 10 feet.

Several smaller test plots were used to compare various jetting techniques and to develop information on the rate at which water should be applied. Also, experiments were carried out using the vibroflotation process. This patented process increases soil density by the penetration of a vibrating tool into the soil. This process, which works better for sandy soils which are more granular than the fine-grained San Joaquin Valley alluvial soils, was not suitable.

The conclusions reached as a result of the experiments carried out at the Mendota test site in summary were:

1. Subsidence causes differential settlements of such severity that canal embankments and linings would be destroyed if the areas were not compacted.
2. Water application is an effective and most economical means of compacting those particular soils subject to subsidence.
3. Gravel-packed infiltration wells increase the rate of subsidence.
4. The extent of cracking measured from the pond's edge is dependent upon the subsidence magnitude and the soil type.

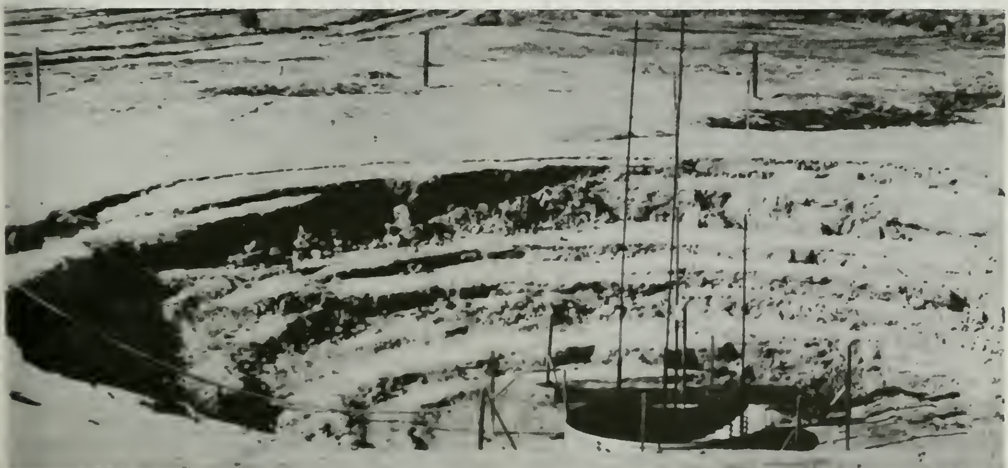

Figure 158. Example of Localized Shallow Subsidence

5. A drying period is required between completion of the ponding operation and construction of the Aqueduct, since the moisture content of soils encountered will be slightly over optimum after a 6- to 12-month drying period.

6. About 80 to 90% of expected shallow subsidence would be achieved prior to construction—allowance would be made for further subsidence by providing extra freeboard.

An additional study and experimental program were developed to explore the potential for soil liquefaction of the hydrocompacted soils during ground motions from earthquakes. A supplemental objective was to determine the dynamic soil characteristics of the canal embankments and foundation soils. This program also was carried out at the Mendota test site.

Phase I of the testing utilized a large vibration generator (40,000 pounds maximum force) to induce a dynamic loading. Resonant frequencies were determined by preliminary testing, and final tests were performed at the resonant frequencies using maximum input forces. Phase II testing utilized explosives to induce dynamic loading. A test pattern utilizing column charges and time-delay blasting caps was developed to simulate a seismic shock in the soil mass. Using this technique, it was possible to induce larger dynamic forces than were attainable with the vibration generator.

Instrumentation for the liquefaction studies was designed to obtain data on embankment vibrations, pore pressures, and vertical and horizontal movements of the test section. The instruments used were geophones, high-speed Brush Recorders, Carlson pore pressure cells, open piezometers, and survey reference points. Some use was made of a Sprengnether portable seismograph and a special strong-motion seismograph operated by the U.S. Coast and Geodetic Survey.

Geophones proved to be the most economical and reliable instrumentation for obtaining vibrational data. Embankment response was measured in three dimensions from several locations simultaneously. The results were compared on an energy-ratio basis with the 1940 El Centro and 1952 Taft earthquakes. These two seismic occurrences were used for comparison as data were recorded from them in deep alluvium and near the earthquakes. Caution was required in making comparisons as maximum amplitudes are more damaging to embankments, whereas duration and frequency of shaking are quite critical in the actual inducement of liquefaction in a soil mass.

The conclusions, based on the liquefaction tests at the Mendota test site, and the analysis of the resulting data follow:

1. Liquefaction of the foundation soils at the Mendota test site was not observed during any phase of the testing.

2. The accelerations caused by the large vibration generator were less than would be expected during a severe earthquake. Duration or cycles of the dynamic loading during this phase of the testing were much greater than would be expected for an earthquake.

3. Accelerations caused by the blasting tests were greater than would be expected for the design earthquake, but the duration of the dynamic loading was shorter than that of a severe earthquake.

4. Compaction, caused by the dynamic loading of a severe earthquake, will result in settlement of the canal embankments in most hydrocompacted subsidence areas. The magnitude of the settlements will be dependent upon the depth of subsidable soils and may be as large as 1 foot for the deeper deposits.

5. Seismic stress of the magnitude induced by the large vibration generator will have little or no effect on the canal embankments.

6. Extensive embankment cracking and settlement will result from seismic loadings of the magnitude induced by the blasting tests. A severe earthquake will cause settlement and cracking of the embankments founded on hydrocompacted alluvial soils.

7. Low-density saturated soils could be compacted in localized problem areas through the use of a blasting technique.

8. No general corrective design is indicated as a result of the liquefaction testing performed at the Mendota test site.

Geology on Canal Alignment

Excavation for the canal was almost entirely in alluvial deposits. Limited stretches of marine rocks (Tejon formation) and continental deposits (Tecuya and Tulare formations) were encountered in the flanks of the Kettleman Hills, in the intake channel to Buena Vista Pumping Plant, in Wind Gap Cut, and in the foothills of the Tehachapi Mountains near A. D. Edmonston Pumping Plant. The various formations encountered by the conveyance system in the South San Joaquin Division are described in more detail in the paragraphs that follow.

The Tejon formation is comprised mostly of massive, gray, silty-sandstone beds with thin interbeds of dark shale. Massive, hard, calcium-carbonate-cemented strata rich in fossils also are encountered. The formation is formed of Eocene marine sedimentary rock and rests unconformably on the crystalline rocks that constitute the core of the Tehachapi Mountains.

The Tecuya formation overlies the marine Tejon sandstones and is of Oligocene-Miocene Age. The formation is comprised of continental deposits, both alluvial fan and shallow water deposits which accumulated at the base of the Mountains near the edge o the retreating Eocene sea. Volcanic rocks, dacite basalt, and agglomerate are interbedded with th sedimentary rocks. An exposure of the Tecuy formation which consists entirely of sedimentar rocks was encountered in bulldozer trenche excavated in the low foothills west of Pastoria Creel The layers are comprised of poorly indurate siltstones and sandy-siltstones and poorl consolidated friable sandstones and conglomerates. The gravels in the conglomerates are comprised of well-rounded coarse-grained boulders and are distinguished by the decomposed coarse-grained rocks.

The Tulare formation is comprised of continental deposits which crop out at widely scattered locations along the west and south sides of the San Joaquin Valley. The formation contains beds of sand, gravel, and mudstone which have accumulated under environmental conditions similar to the present conditions and are, therefore, considered to be alluvial fan and lakebed deposits. The formation is Plio-Pleistocene in age and subsequently has been both folded and faulted. The gravels are limestone, metamorphic rocks, and distinctive white siliceous shale which comes from the Coast Ranges.

Recent Alluvium

A series of coalescing alluvial fans occur at the base of the mountain ranges that border the west and south sides of the San Joaquin Valley. In the central part, the Valley is occupied by lake bottom lands, sloughs, and flat alluvial plains. The alluvial fan deposits accumulated intermittently during brief but intense rainstorms, and the lakebed deposits were transported into the Valley during times of heavy runoff from the Sierra Nevada. At the shoreline of the lakes and on the alluvial fan surfaces, wind-blown deposits are interspersed with alluvial deposits. Character of the soils varies directly with the environment of deposition and the rock types on the alluvium source areas. There are many variations in both the vertical and horizontal distribution of the soil types encountered during the subsurface investigation for the California Aqueduct.

Because the rock types in the source areas are mainly shales and sandstones, the alluvial fan deposits along the west side of the Valley from Kettleman City to Maricopa predominantly are very fine-grained sand and silty sand with lesser amounts of sandy clay, silt, and gravel. Gravels characteristically have flat cobbles of siliceous shale. Interbedded with the alluvial fan deposits are lakebed deposits of silt and clay and fine- to coarse-grained sands and dune sands which were accumulated along the shoreline of ancestral Tulare and Buena Vista Lakes.

Both the San Emigdio Mountains to the south and the Tehachapi Mountains to the southeast of the San Joaquin Valley contain a core of older crystalline rocks that is overlain by northerly dipping, Tertiary, marine and nonmarine, sedimentary rocks and Tertiary volcanic rocks. Alluvial fans at the base of these mountains reflect the greater durability of the crystalline rock types and therefore contain more sand, gravel, and boulders. For this reason, alluvial deposits around the south margins of the San Joaquin Valley are, in general, comprised of coarser-grained soil types.

Adjacent to Buena Vista Slough and along the northern and western edges of Buena Vista Lake, alluvial fan deposits are interbedded with lakebed deposits composed primarily of lacustrine clays and micaceous sands. The bulk of the clay material was transported from the Sierra Nevada by the Kern River. Along the shoreline of the Lake, alluvial fan materials derived from the predominantly coarse-grained Tulare formation in the adjacent Elk Hills and Buena Vista Hills overlie, and are interbedded with, highly plastic clay lakebed deposits. Highly variable deposits have resulted from a fluctuating shoreline and recent uplift of the bordering hills.

Design

The majority of the design features and criteria are consistent with, and in many cases identical to, those used in the North San Joaquin Division, discussed in Chapters I and IV of this volume. The principal differences are the varying dimensions of the canal and the side-slope configuration used for the canal sections in this division.

The canal dimensional changes were determined by the reduction in required capacity between water delivery turnouts within the Division. Changes in physical dimensions were kept to a minimum to facilitate the use of standard operation and maintenance equipment and to reduce construction costs associated with specialized equipment, such as the paving train for the aqueduct lining. The dimensional changes were minimized by changing the depth of water to achieve the required flow.

Flatter side slopes were adopted because of the weaker soils encountered in this division and to allow for residual subsidence from hydrocompaction or foundation liquefaction. Between Tupman Road and Buena Vista Pumping Plant, side slopes of $2\frac{1}{2}$:1 were used because of a combination of high ground water and weak soils.

Freeboard

The operational freeboard of 2.5 feet of lining above normal water surface and 2.5 feet of earth-berm freeboard above the lining is similar to the provisions used in the North San Joaquin Division. However, in this division, a variable contingency freeboard was established to allow for subsidence. Allowance was made for both shallow subsidence from hydrocompaction and deeper subsidence from ground water extraction.

The subsidence freeboard is lined above the operational freeboard, and the amount varied with the foundation conditions encountered. Deep subsidence freeboard varied from zero to 3.5 feet at Wheeler Ridge, where possible future regional tilting also was taken into consideration. Freeboard for shallow subsidence varies from zero to 2 feet and was established from the amount of subsidence experienced in preconstruction consolidation. The total lined freeboard in the Division, therefore, varies from a minimum of 2.5 feet to a maximum of 8.0 feet.

Underdrains

Although ground water during construction was not as extensive a problem as it was in the North San Joaquin Division, it will be a factor during the operation of the facilities in the southern portion of the South San Joaquin Division. Because of the low shear value of the soils over approximately one-half of the 120 miles of canal in this division, an extensive system of underdrains, sumps, and pumps with regulatory equipment was provided to control the ground water level and prevent the soils from becoming saturated.

Severe fluctuations in canal water levels are imposed downstream of project pumping plants by intermittent off-peak pumping. In those locations, design provided ground water level control features consistent with the anticipated canal water fluctuations to avoid overstressing the aqueduct lining.

The underdrains in this division differ from the filter blankets installed in the North San Joaquin Division. Extensive finger drains were constructed using larger size filter material, collector pipes, and automatic permanent pumping equipment.

The finger drains (Figure 159) adopted for the underdrain system are 18-inch-square trench sections placed on the side slopes perpendicular to the centerline of the canal and backfilled with 3-inch-maximum filter material. They are either directly under the concrete canal lining or beneath the compacted sublining on 12-foot - 6-inch to 20-foot centers. The finger drains extend upward for varying distances from 9 feet vertical above the invert to full height. In critical areas, they extend downward from the top of the lining from 7 to 13 feet. The drains were placed on either one or both sides of the canal and

Figure 159. Finger Drain Excavation—Buena Vista Pumping Plant Inlake Channel

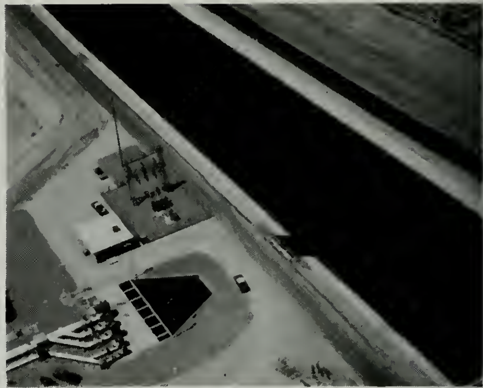

Figure 160. Turnout and Pumping Station for Belridge Irrigation District Near 7th Standard Road

terminate in collector pipes which drain directly or through a header pipe into a sump.

The sumps are 36-inch-diameter reinforced-concrete pipe extending from the top of the lining to approximately 10 feet below canal invert. The collected drainage is pumped from the sumps and discharged into the canal. Pipe cleanouts extend from the header pipes to slightly above ground surface. These cleanouts are located at the beginning and termination of each underdrain siphon and midway between the sumps. The portion of the cleanout extending aboveground is perforated for a vent.

Bridges

Bridge designs were similar to those in the North San Joaquin Division except for locations where further subsidence is expected. At these locations design provided for raising the structure by jacking

Turnouts

Turnouts (Figure 160) were classed as major o minor. Major turnouts are those with the capability o delivering 200 cfs or 5% of the aqueduct flow with th water surface at the minimum operating level. Mino turnouts are those with a lesser delivery capabilit All turnouts in the South San Joaquin Division no are being equipped with automated flow contro operated either locally or from the area control cente

Turnouts were designed for either gravity or pum delivery with design standardized where possibl Special conditions, however, required individu design. In some cases, the design was prepared by th water users. In all cases, the design required mutu approval. If possible, the turnout construction w incorporated within the aqueduct contracts or b grouping the turnouts into a separate contract. I cases where the details of the turnout were n available but the location was known, only th headworks and a short section of delivery pipe we constructed. If the location was selected after lining construction, the lining was cut for the installation.

Turnout headworks normally consist of a trashrack with slide gates and a supporting structure. In some cases, the slide gates will not be installed until the water user constructs the connecting delivery system. Stoplogs are provided for dewatering the turnout. Deliveries are measured by a flow tube for a pipeline system or by a Parshall flume for an open-channel system. The slide gates are of the flat-back type and were designed to operate against a seating head of 20 feet and an unseating head of 5 feet.

The measuring system consists of primary and secondary equipment. Flow tubes and Parshall flumes are primary equipment; the secondary equipment provides a flow rate and records of flow. Flow through a Parshall flume is determined by the relationship between the width of the throat and the height of water in the stilling well. Secondary equipment converts the water-level reading from the stilling well to a flow-rate signal. Rate of flow through a flow tube is determined by the difference in pressure of the water at the inlet and throat sections of the tube. Secondary equipment converts the difference in the water pressure into a flow-rate signal. In both cases, the secondary equipment provides the on-site instantaneous flow rate, a readout of the totalized flow, and a recorder chart showing the continuous flow pattern.

Construction

Construction was supervised from a project office in Bakersfield. Field offices were established as needed at construction sites. A soils and concrete laboratory was established at Taft.

Because of the lead time required between preconsolidation and actual canal construction, work commenced on the preconsolidation contracts as early as the summer of 1963. The last section of the conveyance facilities was completed in the spring of 1971. The description of aqueduct construction is presented in a north-to-south order, irrespective of contract dates. The preconsolidation contracts are described first, since the preconsolidation work necessarily preceded aqueduct construction and was distinctive in itself.

Throughout the period of construction of the open-channel aqueduct in the South San Joaquin Division, new contraction joint designs, new methods of sealing the joints, and new sealants were proposed, investigated, and some were approved.

Preconsolidation Contracts

There were six major construction contracts covering five locations for preconsolidation ponds. There were two supplemental contracts for water wells and pumping equipment. Fifteen service and supply contracts were utilized to provide such back-up services as power facilities, pipe distribution systems, and contracts to furnish ponding water.

General information about the six major preconsolidation contracts is shown in Table 12.

Vicinity of Arroyo Pino Creek. This contract included three-tenths of a mile of the Aqueduct just south of Kettleman Hills, which was the only portion where significant shallow subsidence was identified in the first 39 miles of the South San Joaquin Division. The work consisted of constructing four consolidation ponds with turnouts and necessary water delivery pipelines and appurtenances. One of the ponds was in the channel of Arroyo Pino Creek.

The ponds were excavated and the surrounding dikes constructed from the excavated material with conventional earth-moving equipment. The ponds within the runoff channel were leveled with an uncompacted layer of earth to provide a uniform

TABLE 12. Major Preconsolidation Contracts—South San Joaquin Division

	Specification	Low bid amount	Final contract cost	Total cost—change orders	Starting date	Completion date	Prime contractor
Arroyo Pino Features Mile 177.4 to Mile 177.7	65-16	$13,602	$16,972	--	5/17/65	7/ 1/65	W. M. Lyles Co.
Lerdo Highway to Tupman Road Mile 215.6 to Mile 238.9	64-41	2,213,629	2,354,106	$31,662	11/18/64	11/10/65	Peter Kiewit Sons' Co.
Buena Vista Pumping Plant to Wheeler Ridge No. 1 Mile 255.7 to Mile 279.2	64-46	3,910,386	4,184,121	135,756	1/ 2/65	3/11/66	Eugene Luhr & Co. and Hood Corporation and Hood Construction Co.
Sunset Railroad to Maricopa Highway Mile 261.6 to Mile 274.3	66-12	565,410	558,936	2,438	4/ 5/66	7/29/66	William H. Schallock
Wheeler Ridge Pumping Plant No. 1 to Standard Oil Company Road Mile 279.2 to Mile 283.9	64-21	874,672	796,403	54,041	6/22/64	4/ 2/65	William H. Schallock
Standard Oil Company Road to Grapevine Creek Mile 283.9 to Mile 288.7	63-32	610,539	635,092	37,949	11/ 8/63	3/31/65	Pascal and Ludwig

surface for the water application.

The water for preconsolidation was furnished and applied by the Tulare Lake Water Storage District. This district is a user of project water and, by agreement, was reimbursed in kind from the completed California Aqueduct for the water used. This arrangement was frequently followed for the preconsolidation contracts. Ponding operations began on July 7, 1965 and terminated on October 4, 1965. Only minor subsidence occurred, about 0.7 of a foot.

Lerdo Highway to Tupman Road. This contract extended over 23.3 miles and consisted of 273 subsidence ponds with infiltration wells. This section of the Aqueduct parallels the West Side Canal of the Buena Vista Water Storage District. Arrangements were made with this district to supply the ponding water. The contract included installation of the necessary pumps, motors, pipelines, and appurtenances to deliver the water from the West Side Canal to the filtration ponds.

The filtration ponds (Figure 161) usually were 200 by 500 feet but were widened as required to treat bridge and drainage structure sites. Excavation and dike compaction were conventional. The contractor used newly designed 32-cubic-yard scrapers. He also used a sprinkling system to wet the excavation areas prior to removal of the material.

A typical pond contained 14 gravel-packed infiltration wells without benchmarks and one well with a benchmark for measuring the subsidence at the bottom of the well. The depth of the wells varied from 40 to 80 feet, depending on information obtained during the foundation investigation program. In some cases, ground water was encountered prior to reaching the specified depth. If ground water was reached within 10 feet of the specified depth, the well was terminated at that depth.

Figure 161. Subsidence Ponds in Preconsolidation Area

Figure 162. Typical Cracking Caused by Preconsolidation of Subsidence Areas

The infiltration wells were drilled with auger equipment operated from motor cranes. Soil samples from each hole were tested for later use in design of the Aqueduct. The gravel was fed into the holes by means of an adjustable tremie operated by a drilling crane. The discharge end of the tremie was maintained 1 foot above the gravel level. Gravel was mounded above the hole and covered prior to water application.

Water for the ponds was obtained by installing pumps at suitable locations along the West Side Canal. The pumps were set on pile-supported platforms with the necessary delivery, header, and pond pipes running from each pump site.

Several road detours and service roads were included in the contract. Department personnel applied the water to the ponds. Work commenced in June 1965 and was completed in April 1967. A total of slightly more than 45,000 acre-feet was applied to the ponds. Maximum surface subsidence was 1.23 feet.

Buena Vista Pumping Plant to Wheeler Ridge Pumping Plant. This contract extended over 23.5 miles of the aqueduct alignment. Over the northerly quarter and southerly half of this alignment, medium to high subsidence was anticipated. However, over a length of about 7 miles from the Maricopa Road crossing westward, the anticipated subsidence was uncertain. The degree of treatment was determined by excavating a "V" ditch along this length and placing infiltration wells on 500-foot centers. The resulting subsidence demonstrated the need for further treatment which was provided by a later contract. Figures 162 and 163 show typical cracking in this reach.

Figure 163. Subsidence Cracks at Cadet Road

A total of 190 ponds was constructed along the remaining alignment. Water for these ponds was supplied and delivered by Buena Vista Associates to a 6-acre-foot holding reservoir from which a battery of six 800-horsepower multistage pumps delivered the water through feeder lines to the ponds (Figure 164).

From Santiago Creek to the Maricopa Road crossing, the aqueduct alignment intersected farm rrigation lines which were temporarily rerouted across the aqueduct alignment on compacted utility ramps during preconsolidation and canal excavation. Surface irrigation flows were collected in ditches at he right-of-way line and returned to the various rrigation systems by pumping through the rerouted ines. Altogether, 38 irrigation lines were serviced rom 15 ditches. Later, permanent relocations were nade by lines placed under the Aqueduct.

Construction methods and equipment were similar o those used in the preconsolidation areas to the north.

The application of water to the ponds began on September 1, 1965 and ended on March 10, 1968. A otal of almost 62,000 acre-feet of water was used, and ubsidence up to 9 feet was observed near the Sunset Railroad crossing.

Sunset Railroad to Maricopa Highway. Seventy-our ponds with infiltration wells were constructed long the "V" ditch of the prior preconsolidation ontract. The temporary detours and preconsolid-on ponding for the permanent siphon crossings of he Sunset Railroad and the Basic School Road were ncluded in this contract. Two other detours for road rossings also were constructed at Old River and Maricopa Roads.

Construction methods were similar to those used on previous preconsolidation contracts. Water deliveries of 11,438 acre-feet were made between August 11, 1966 and March 10, 1968. Maximum subsidence at the Sunset Railroad section was 7.5 feet.

Wheeler Ridge Pumping Plant to Standard Oil Road. This contract extended 2 miles beyond the Wind Gap Pumping Plant site. The ponds and infiltration wells were constructed in the same manner as those for the earlier contracts.

In the Wind Gap area, preconsolidation water was applied by means of a sprinkler system rather than by ponding. This method was chosen in preference to ponding because the rough terrain in this area would have required excessive amounts of excavation with attendant high costs. The water was obtained from wells at Grapevine and Tecuya Creeks which had been drilled under a previous contract. The welded-steel pipe conveyance line included a pressure-regulating station and two pressure-relief stations.

Between August 25, 1964 and April 11, 1968, a total of 6,197 acre-feet of water was applied. Subsidence was minor except at the northern and southern ends of the contract. Maximum subsidence was 5.4 feet and occurred at the Wheeler Ridge Pumping Plant site.

Standard Oil Road to Grapevine Creek. This 5-mile reach was the southernmost preconsolidated section and extended 1½ miles west of the aqueduct undercrossing at U.S. Highway 99.

Water was obtained from the two wells used for the contract immediately to the north. The application of water to the ponds began on August 25, 1964, with a total of 6,303 acre-feet being used. The maximum subsidence was 1.7 feet, about 1 mile upstream of the Highway 99 crossing.

Figure 164. Delivery Basin Between Buena Vista and Wheeler Ridge Pumping Plants

Aqueduct Contracts

There were six major contracts for the construction of the conveyance facilities within the South San Joaquin Division. General information about these contracts is shown in Table 13. In addition to the aqueduct contracts, there were two contracts for turnouts between Kettleman City and 7th Standard Road and between Tupman Road and A. D. Edmonston Pumping Plant.

Aqueduct construction (1965–71) proceeded concurrently with preconsolidation in certain instances. In some cases, the drying time was insufficient for soils to lose enough moisture for optimum compaction. In these cases, blading, windrowing, and manipulation of the soils were required until the proper compaction moisture content was obtained.

The heavy rains of the 1968–69 winter also delayed construction and partially damaged completed features of the conveyance system.

Kettleman City to Avenal Gap

Design. This section of the Aqueduct is 12.3 miles long and begins at the end of the transition from Check No. 21, just north of Kettleman City. The design capacity changes from 8,350 cfs to 8,100 cfs at this check. This section ends at the transition to Check No. 22 just downstream from Avenal Gap Siphon and the Coastal Branch turnout.

The canal is 32 feet wide at invert, side slopes are 2:1, and total freeboard is 5 feet. Most of the canal prism was in cut and closely follows the 300-foot-elevation contour.

The aqueduct alignment in this section traverses the northeastern flanks of the North and Middle Domes of the Kettleman Hills which are drained by small intermittent streams.

Near Avenal Gap, low-density gypsiferous clay and silt were encountered with gypsum concentrations approximating 53%. Removal of these soils and replacement with a compacted sublining were specified. Sulfate-resistant concrete was required in the lining. Just upstream of Avenal Gap, for 300 feet, potentially expansive clay was specified for removal and replacement with a compacted sublining.

Ground water was not a problem. Subsurface exploration indicated that the closest ground water was near Avenal Gap and 48 feet below canal invert. The contractor drilled two dry holes near the middle of the contract area to 1,000-foot depths in an attempt to develop construction water.

The main features in this contract were: 12.3 miles of lined canal, three county road crossings, two oil-company road crossings, one operating road crossing, one siphon at Avenal Gap, five culvert and nine overchute drainage structures, and six turnouts including the Coastal Branch turnout. No checks are located in this construction reach. In addition to department-constructed bridges, the Division of Highways constructed bridges for State Highway 4 and Interstate Highway 5 overcrossings.

Design standards of these features conformed to those for the previous aqueduct contracts.

The Coastal Branch turnout is a double-barreled rectangular, weir section. The 7-foot by 8-foot barrels have provision for stoplogs.

TABLE 13. Major Aqueduct Contracts—South San Joaquin Division

	Specification	Low bid amount	Final contract cost	Total cost—change orders	Starting date	Completion date	Prime contractor
Aqueduct—Kettleman City to Avenal Gap Mile 174.8 to Mile 187.1	66-03	$9,108,350	$10,155,801	$193,624	2/17/66	12/12/67	Gordon H. Ball Enterprises and Granite Construction Co.
Aqueduct—Avenal Gap to 7th Standard Road Mile 187.1 to Mile 220.1	65-28	20,883,720	23,398,824	320,169	7/24/65	5/29/68	Gordon H. Ball Enterprises and Granite Construction Co.
Aqueduct—7th Standard Road to Tupman Road Mile 220.1 to Mile 239.0	67-06	10,482,625	10,560,006	54,288	4/13/67	6/12/69	Peter Kiewit Sons' Co.
Aqueduct—Tupman Road to Buena Vista Pumping Plant Mile 239.0 to Mile 251.8	67-37	10,465,422	11,058,419	66,219	9/ 2/67	9/23/69	Western Contracting Corporation
Aqueduct—Buena Vista Pumping Plant to Wheeler Ridge Pumping Plant Mile 251.8 to Mile 280.0	67-69	19,975,006	20,398,353	144,076	1/22/68	8/31/70	Griffith Company
Aqueduct—Wheeler Ridge Pumping Plant to A.D. Edmonston Pumping Plant Mile 280.0 to Mile 295.8	68-07	12,685,693	13,464,810	312,803	5/10/68	4/15/71	Griffith Company

Avenal Gap Siphon does not include an upstream check structure. However, stoplog provisions are included both upstream and downstream of the Siphon. The siphon barrel consists of three 18-foot-square sections 187 feet long. A pile-supported operational bridge is located on the left side of the Siphon. The channel is paved with a 6-inch reinforced-concrete apron placed on a 6-inch blanket of filter material. Three longitudinal, 6-inch-diameter, perforated-pipe underdrains are bedded in the filter material. The concrete apron extends nearly 100 feet on both sides of the centerline of the Siphon and has cutoff walls and riprap scour protection at both ends.

Construction. Earthwork for the canal prism was done by conventional earth-moving equipment. Considerable moisture conditioning of the embankment materials was required because of their low, natural, moisture content and because of high daily temperatures. Downstream from Avenal Gap, in the more highly consolidated portions of the Tulare formation, overexcavation and backfill with a compacted subliner were required in addition to the sublining for excess gypsum and expansive clays in a manner similar to that used at and upstream from Avenal Gap.

Because no underdrain system was required, the canal trimming operation was simplified. The lining was placed with equipment trains of batch-fed pavers, finishers, and curing jumbos. Polyvinyl waterstop was installed in the transverse and longitudinal grooves.

Avenal Gap Siphon was constructed by first placing the floor slab and then placing the exterior walls, interior walls, and top slab monolithically in 22-foot-long sections.

The following year, when the turnouts were completed, the water users also were completing their distribution systems. This planned phasing permitted earlier water delivery. Seventy-two-inch slide gates were specified for all turnouts.

Water first flowed into this section of the Aqueduct on January 13, 1968. The first water was delivered to the Dudley Ridge Water Storage District on February 1, 1968.

Avenal Gap to 7th Standard Road

Design. The contract for this 33.9-mile section was the longest on the California Aqueduct. This portion of the Aqueduct passes along the southeastern flank of South Dome of the Kettleman Hills and follows the eastern flank of Antelope Plain between Kettleman Hills and Lost Hills to the Lost Hills anticline.

The Aqueduct in this area is nearly all in cut. The drainage channels crossed are not sharply defined but are broad depressions. The Tulare formation occurs extensively. In addition to soils with a high gypsum content and potentially expansive clays, cohesionless, fine-grained, low-density soils lacking shear strength when saturated also were encountered. In all cases where these conditions existed, the specifications required 3-foot overexcavation normal to the canal prism backfilled with an impervious compacted sublining. This sublining was required for 80% of the contract reach.

Ground water was not a problem. Wells 400 to 600 feet deep were required to obtain construction water. The canal prism has the same 32-foot bottom width and 2:1 side slopes as the section to the north. However, the design capacity changes from 8,100 cfs to 7,150 cfs at the check structure at the head of this reach. The design capacity further reduces to 6,350 and 5,950 cfs at checks 25 miles downstream and at 7th Standard Road. The total freeboard increases southward to allow for greater subsidence, changing from 6.1 feet at Avenal Gap to 7.5 feet at 7th Standard Road.

In addition to the lined channel, the reach constructed under this contract contained 4 check structures, each with 4 radial gates; 1 canal drain just upstream of 7th Standard Road; 4 county road bridges and 3 operational bridges; 3 culverts and 25 overchutes; and 11 irrigation turnouts. No siphons were needed in this section. The Division of Highways constructed an overcrossing for State Highway 46.

The canal drain which discharges into the Kern River flood control channel is a two-barrel, 48-inch-diameter, pipe structure and is gate-controlled from an access shaft adjacent to the operating road. A trashrack is incorporated into the entrance structure. A 36-inch reinforced-concrete pipe provides local cross drainage under the aqueduct prism at this station and joins the canal drain at a small stilling basin 75 feet from the gate-access shaft. The drain then joins a 10-foot-bottom-width canal with 2:1 side slopes extending at right angles to the Aqueduct for over 1 mile to the Buena Vista Lake flood control channel. Six drop structures in the channel accommodate the 32-foot drop in elevation.

Construction. The prime contractor for this reach also constructed the reach to the north and moved equipment between the two contracts.

The excavation and embankment operations were noteworthy for the extensive overexcavation and moisture conditioning required. The trimming equipment and paving train (Figure 165) were the same as used on the reach to the north. However, up to the first check downstream from the start of the contract, the longitudinal and transverse contraction joints were struck by a knifelike instrument immediately behind the paver and the joint held open with a plastic insert placed in the joint. The plastic inserts were removed prior to sealing the joints.

Beyond the first check, a polyvinyl waterstop was used for the longitudinal joints and was inserted when the concrete was placed. The transverse-joint detail was not changed.

The length of the contract imposed logistic difficulties with the structural concrete. With 25

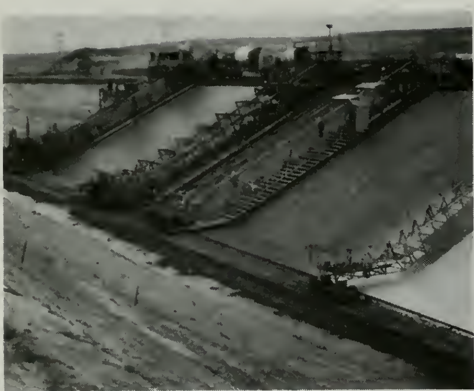

Figure 165. Paving Train

Figure 166. Check Structure During Construction

Figure 167. Buena Vista Turnout (Buena Vista Outlet Canal in Background)

overchutes receiving seven placements each and spread throughout 32 miles, careful scheduling and support were required. On one day, 17 separate placements were made. The achievement of proper curing by water for the required ten days was particularly critical, especially during the hot months, and required a seven-day around-the-clock effort. Because of the larger quantities involved, placements for bridges and checks were more easily accomplished.

Nine of the eleven turnouts are slide-gate-controlled. The other two are open-pipe diversions to pumping plant forebays. Six of the gated turnouts are hand-operated. Individual characteristics of the turnouts and the number of embedded parts required several placements and intricate forming.

7th Standard Road to Tupman Road

Design. For much of the 17-mile length of this reach, the Aqueduct parallels the flood channel of the Kern River. As the aqueduct alignment approaches Tupman Road and Buena Vista Lake, the level of ground water rises; however, it was still sufficiently below the elevation of the canal invert to negate the need for uplift-pressure control. The soils continued to vary in composition within narrow boundaries, and high-gypsum, low-density, and expansive clay conditions again were encountered and corrected by overexcavation and backfill with select compacted materials. The canal prism retains the dimensions of the section to the north; however, the design capacity changes at the start of this reach from 6,350 to 5,590 cfs and remains unchanged to Buena Vista Pumping Plant, 32 miles downstream.

Essentially, the design of the features in this reach was the same as for the other contracts. This reach comprises 17 miles of lined aqueduct with primary and secondary operating roads, six bridges including two operation bridges, and four county road overcrossings. The Division of Highways constructed an overcrossing for State Highway 58. There are two radial-gated check structures (Figure 166) with the number of bays now reduced from four to three because of the decreasing design capacity of the Aqueduct. There are 15 overchutes and 1 siphon beneath Temblor Creek. There also are two turnouts and several pipe crossings of the Aqueduct.

The cross-drainage structures are distinctive in this division. No drainage culverts were used, only overchutes. Temblor Siphon is similar to Avenal Gap Siphon in that it is an unchecked three-barrel structure. This siphon has 16- by 16-foot barrels 30[illegible] feet long. Under maximum runoff, the flow in Temblor Creek will be a low-velocity sheet flow, and riprap or concrete-slab protection was determined unnecessary for scour protection. A pile-supported operating bridge spans the Creek at the Siphon.

There are two turnouts: a small, 20-cfs, 24-inch pipe-inlet turnout with a hand-operated slide gate and a 250-cfs turnout (Figure 167) near Tupman Road

The latter turnout consists of a 7- by 7-foot entry culvert with a hydraulically controlled slide gate, a Parshall flume, and control building. The 1968–69 Kern River floodflows were admitted through this turnout into the Aqueduct.

Construction. The contractor was experienced on the Aqueduct, having been the contractor on a reach in the North San Joaquin Division.

This contract commenced when the last water was applied to the preconsolidation ponds at the southern end of the contract. Since the soils had insufficient time to dry to an optimum moisture content, the contract specifications instructed the contractor to schedule his activities around the locations between Elk Hills Road and Tupman Road until other areas became available.

Excavation commenced at the north end of the contract and moved southward. Prewetting was not required because of moisture remaining from preconsolidation. The top 5 to 10 feet of the prism was excavated first with rubber-tired power scrapers. When the soil was too wet for this type of equipment, the equipment was moved to a drier location. However, in many instances, the soil in the lower elevations of the prism remained too wet, and track-mounted equipment was required to complete the excavation. Excavation for the bridges occurred early in the contract because specifications required early abandonment of the temporary detours established during the preconsolidation contract.

In most areas where fat clays occurred, they were overexcavated; however, for a section near Tupman Road, these clays were so saturated that they were left in place within 2 feet of final grade until just before lining the canal prism. This procedure had been used previously in some locations in the North San Joaquin Division. Early rains complicated excavation. Normal overexcavation of soils with a high gypsum content or low-density cohesionless soils continued. One small slide (about 7,000 cubic yards) occurred where preconsolidation water had not fully dissipated, thus creating sufficient pore pressure to cause the slide.

Due to the low-density soils, most structure foundations required overexcavation and filling with compacted embankment. All bridge abutments and piers were supported on piles. Other structures were supported by spread footings.

The trimming and lining equipment previously had been used by the contractor in the North San Joaquin Division. The equipment was modified for the smaller prism in this reach. The lining was placed in three sections: first, a 10-foot invert strip and the adjacent slope lining; then, a similar section of lining on the opposite side of the canal; and, finally, the 12-foot center section of the invert. Flat 2½-inch waterstops were used in the longitudinal joints.

In spite of heavy rains during two winter seasons, the contract was completed three months ahead of schedule. Water was admitted into each reach between checks as soon as construction permitted.

Tupman Road to Buena Vista Pumping Plant Intake Channel

Design. This 15.2-mile reach of the Aqueduct follows the western edge of Buena Vista Lake to the intake channel of Buena Vista Pumping Plant, the first pump lift beyond Dos Amigos Pumping Plant in the San Luis Division.

The terrain is flat with little, if any, relief. Cross drainage from the west is local with depression washes and no defined stream channels. To the east, from Tupman Road to the existing shoreline of Buena Vista Lake, the canal alignment intermingles with the somewhat meandering path of the Buena Vista outlet channel. This channel serves as the irrigation outlet for Kern River waters stored in Buena Vista Lake. The channel was relocated to the northeast of the aqueduct alignment.

Overexcavation and compacted backfill were required wherever low-density soils and expansive clays were encountered or where the gypsum content of the soils was 10% or greater. During construction, soils laboratory personnel developed a rapid field test for gypsum content which aided considerably in identifying the areas requiring overexcavation.

The aqueduct prism is nearly all in cut. The ground water table rises steeply from 50 feet below invert elevation at Tupman Road to 12 feet above invert at Buena Vista Lake. Because of this, an underdrain system was provided. This system, as explained earlier in this chapter, employed finger drains rather than the continuous blanket of filter material used in the North San Joaquin Division.

Local influence on the ground water level between Buena Vista Lake and the Aqueduct during high-water periods in the Lake required special control. For a distance of 2 miles downstream from Tupman Road, where the aqueduct alignment is close to either the lakebed or the Buena Vista outlet channel, a special underdrain system was developed during construction.

The system was installed only on the left or lake side immediately below the lining. The finger drains are on 12-foot - 6-inch centers and extend along the side slopes from the invert, a vertical distance of 9 feet. Because the system was needed only during periods of high water in the Lake, a permanent pumping system was not included.

Ground water was of poor quality with up to 87,500 parts per million dissolved solids. Therefore, to avoid discharging any of this poor quality water on adjacent downslope areas where it would be detrimental to agriculture, all ground water was diverted to evaporation ponds for dissipation. These ponds were constructed adjacent to the canal prism on the upslope side.

The evaporation ponds were not interconnected and varied in length from 100 to 1,100 feet and in width from 100 to 240 feet. Normal embankment was specified for the dikes with side slopes of 2:1 and a top width of 12 feet. In some of the larger ponds, interior dikes containing an equalizer pipe were specified for erosion control.

The Aqueduct in this reach has the same configuration and capacity as the reach to the north except, because of the less favorable soils present, 2½:1 side slopes were adopted. Auxiliary features included six farm or utility access bridges, one operational bridge and two utility pipeline crossing bridges, two three-bay check structures, nine overchutes, and the intake structure for one single-barrel turnout. Under separate contract, this turnout was completed and three additional turnouts constructed. The Division of Highways, under separate contract, constructed an overcrossing for State Highway 119. There are no siphon structures and, except for State Highway 119, no other road crossings.

Six of the overchutes are distinctive to the Project in that they dispose of the drainage by pumping the flow across the Aqueduct. In the aqueduct reach between Tupman Road and the inlet of the Buena Vista outlet channel, the local drainage from Elk Hills and other flood waters are collected in sump inlets, pumped across the Aqueduct, and discharged into the adjacent Buena Vista outlet channel. Each of the installations consists of a reinforced-concrete inlet structure with a grated opening and a section of corrugated-metal pipe leading to a corrugated-metal tee section. This latter section acted as a small sump and contained the suction tube for a vertically mounted pump. The pump discharges into a steel pipe beneath the secondary operating road crossing the Aqueduct on a single-pier support. A corrugated pipe then conveys the flow under the primary operating road and into the Buena Vista outlet channel through rectangular, reinforced-concrete, outlet structures. The pumps are automatically controlled from water-level sensors mounted in the wall of the corrugated tee section and are sized to dissipate the ponded flows that would result from a 500-year storm, within a week's time.

Construction. The contractor had previously worked on the North San Joaquin Division conveyance facilities and was familiar with project specifications.

Since contract specifications required that the Buena Vista outlet channel be available during irrigation seasons for water distribution, the relocation of the channel was accomplished during the first winter season of the contract.

Because of storage of floodflows in Buena Vista Lake, the specifications alerted bidders to the possibility of an unusually high water content in the soils. Throughout much of the aqueduct alignment, inspection pits were dozed every 1,000 feet by the contractor. This procedure furnished information on the amount of prewetting which would be required.

Excavation and embankment placement generally were accomplished with rubber-tired equipment, and only in a limited number of cases was track equipment required. In the area of the regular underdrain system, the canal was excavated by dragline to within 1 foot of invert elevation. The dragline also was used on several other short sections of canal.

A trenching machine excavated the trenches for the pipe underdrain system. The slots for the finger drains were excavated by backhoes. A belt loader fed from a traveling hopper supplied with material by bottom-dump trucks was used to place the filter material, and mechanical vibrators were used to evenly distribute the material. The top section of the sump manhole was placed after the lining because it projected above the lining. As soon as the underdrain system was installed, the poor-quality water was pumped into the evaporation ponds on the right bank of the canal prism (Figure 168).

The canal trimming and lining equipment had been used satisfactorily on other project contracts and

Figure 168. Ground Water Seepage Into Aqueduct Excavation—Installation of Underdrains in Progress

Figure 169. Floodflow Discharge Ponds

Figure 170. Completed Turnout With Water in Aqueduct at Near-Operating Level

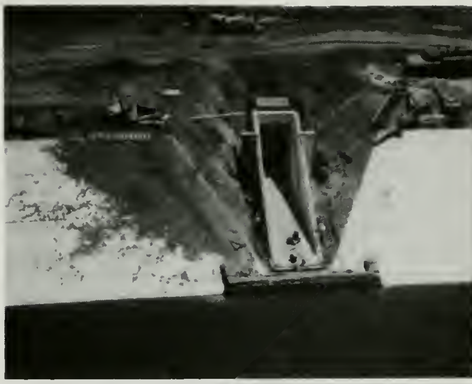

Figure 171. Turnout Construction in Progress After Canal Lining Had Been Completed

required only minor modifications for this contract.

During the heavy rains of 1968–69, the rising water level in Buena Vista Lake resulted in water seeping into the canal. Because the lining had been completed north of the State Highway 119 overcrossing, a sandbag dike was constructed in the Aqueduct upstream of the bridge and the Aqueduct filled with 4 feet of water to balance uplift on the lining. The underdrain system also was activated, which lowered the ground water level below the invert elevation.

The underdrain system farther downstream and adjacent to Buena Vista Lake also was activated but did not operate as satisfactorily as the upstream system, probably for two reasons: (1) the soils downstream contained beds of clay which prevented or hindered the movement of the ground water, and (2) gypsum crystals formed in the perforations of the collector pipes and plugged them.

The contractor drove piles or placed footings for the crossing structures and then lined the Aqueduct before building the superstructures to avoid removing and reassembling the trimming and lining equipment at each crossing. Bypass channels also were excavated around the two check structures to provide for the passage of equipment.

Dispersion ponds (Figure 169) were constructed on the Buena Vista Lake side of the Aqueduct to receive the flows from the three overchutes adjacent to the Lake. The dispersion ponds were similar in construction to the evaporation ponds, with the exception that a series of weirs were incorporated in the earthen dike embankment on the lake side of the ponds. The crest of these weirs is 3 feet below the top of the dike, or 5 feet above the pond invert elevation. All weirs (10 each in two of the ponds and 12 in the third) in each pond are set to discharge simultaneously.

The single, turnout, intake structure (Figure 170) in this contract is a 60-inch-diameter pipe controlled by a 60-inch-square slide gate with manual and automatic hydraulic operators. Under the later turnout contract, the pipe was extended 168 feet.

The other three turnouts are slide-gate-controlled intake structures with short lengths of pipe temporarily bulkheaded at the discharge end. The intake structure has provisions for stoplogs to dewater the turnout. The turnouts were completed later by the water users (Figure 171).

A control building with operating and measuring equipment and a short access road to the control building also were constructed.

Buena Vista Pumping Plant to Wheeler Ridge Pumping Plant

Design. This 26.8-mile reach is the southernmost portion of the California Aqueduct on the valley floor, upstream of the foothills of the Tehachapi Mountains.

The alignment follows the 500-foot-elevation contour around the southeast end of the Buena Vista Hills through gently dipping beds of the Tulare formation, across the alluvial fill of synclinal Midway Valley, and then southeastward across Maricopa Flat to the base of the Wheeler Ridge extension of the Pleito Hills of the San Emigdio Coast Range.

The major features of this contract reach, besides the lined canal and 2 underdrain systems, are siphon undercrossings, 3 of which contain integral, upstream, 3-bay, radial-gated, check structures; 1 road siphon undercrossing and 1 combined road and railroad siphon undercrossing; 1 separate, 3-bay, radial-gated, check structure; 1 spill basin upstream of Buena Vista Pumping Plant; 11 bridges consisting of 4 county road overcrossings, 1 farm road overcrossing, and 6 operational bridges; 2 sediment traps, one each upstream of Buena Vista and Wheeler Ridge Pumping Plants; 8 overchutes; and several culvert drainage undercrossings. There are 10 turnouts in this reach and 9 were constructed under this contract. An overcrossing of State Highway 166 was constructed by the Division of Highways.

All siphons were designed as uniformly as possible to minimize construction costs. All are 16-foot-square three-barreled structures, varying in length from 189 to 393 feet. They are similar to culverts since they have very little sag or drop in invert elevation. The longest siphon, which crosses under the Sunset Railroad and the Basic School Road, has a level gradient. Siphons without checks have identically dimensioned upstream and downstream stoplog supports, and inlet and outlet transitions. At the creek siphons, scour protection was provided by a 6-inch layer of filter material overlain by an 18-inch layer of stone.

The underdrain system upstream of Buena Vista Pumping Plant consists of 18-inch-square finger drains on 20-foot centers extending from the top of the lining to invert elevation. Six-inch perforated pipes embedded in 18-inch by 24-inch filter drains running longitudinally along each side of the invert act as collectors for a sump in the right side of the invert 2,500 feet from the downstream end of the system. An automatically controlled, submersible, sump pump is provided with access through a box manhole at the top of the lining. The system is vented at two locations on the left side of the Aqueduct by 6-inch pipe with watertight joints. The vent pipe extends across the canal invert and connects with and vents both of the longitudinal collector pipes.

The drain system downstream from Buena Vista Pumping Plant (Figure 172) has similar finger drains

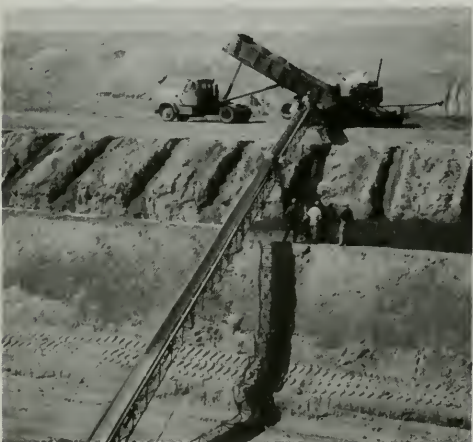

Figure 172. Placing Filter Material for the Underdrain System

in only the upper 7 feet of the lining on each side. The system equalizes water levels in the canal and behind the lining during drawdown. The connector pipes from the drains are 4 inches in diameter and lead to 6-inch pipes which drain the water into three 36-inch reinforced-concrete pipes spaced longitudinally along the system. These pipes act as sumps and are pumped by electric submersible pumps.

The spill basin just upstream of the Buena Vista Pumping Plant is a small 325-acre pond which absorbs surges in water levels should extreme rejection of pumping load occur which might cause overtopping of the canal lining. The pond is formed by an earth dike with a top width of 20 feet. Spills enter the basin over a 300-foot-long weir section on a depressed section of the aqueduct embankment. The crest of the weir is set 1.5 feet above design water surface. Any spilled water must be pumped back into the Aqueduct with portable pumps.

Sediment traps upstream of the pumping plant forebays are comprised of three cells on each side of the centerline beneath the aqueduct invert. The traps are rectangular in shape, 6 feet deep, 48 feet long, and 11 feet - 3 inches wide. Lengthwise, the trap is partially open to the flow and is divided into three sections. The first quarter is open without any restrictions, the second quarter is covered with a grizzly of 3-inch channels on 8-inch centers perpendicular to the flow, and the final half of the trap is covered with 6-inch concrete slabs. Since the need for sediment removal was expected to occur infrequently, no provision was made in the design for hydraulic or mechanical removal of sediments contained by the traps. Sediment removal will be done by maintenance forces using portable equipment.

The overchutes were similar in design to those in the other contract reaches. Training dikes for cross-drainage control are provided at two locations.

The turnout just upstream of the Buena Vista Pumping Plant forebay is a single-barrel, 42-inch reinforced-concrete pipe with flow tube. The inlet is a standard, hydraulically controlled, gated structure. The other turnouts vary from 36-inch-diameter, single-barrel, entrance structures to a 60-inch-diameter double-barrel structure. Three of the turnouts have metal slide gates, one automatically controlled. The remaining turnouts have stoplog provisions for dewatering.

An instrument building with measuring and operating equipment and an access road are part of these installations.

Construction. The contractor for this reach constructed reaches of the North San Joaquin Division and was familiar with department specifications. Much of the equipment used was the same equipment used in the North San Joaquin Division.

Excavation was by scrapers and push dozers. The ground at several locations within the preconsolidation area was too wet for continuous excavation. The contractor in this situation excavated the upper layer as far as possible and then scarified the wet soil to hasten drying and continued this sequence as often as necessary. At invert elevation in areas of preconsolidation, the ground surface was scarified to a depth of 12 inches and thoroughly compacted with heavy rollers prior to the placement of any embankment soils for the foundation grade.

Excavation for the various structures including the siphons was, where possible, accomplished at the same time as canal excavation. This excavation generally required overexcavation and replacement with compacted backfill.

The overchutes and pipeline crossings had separate footings for the abutment and pier sections. The siphons were placed in 30-foot-long sections with a 6-inch waterstop and 1 inch of expansion joint material providing a watertight joint. In this section of the Aqueduct, the bearing resistance of the soils was sufficient to permit the use of footings on all bridges which were of the dropped-stem girder design. The bridges were scheduled by specification for early completion and were completed prior to the trimming and lining operations.

The trimming and lining equipment was the same equipment another contractor had used on the 7th Standard Road to Tupman Road contract. The contractor became quite adept at moving the trimming and lining equipment past a structure, often completing the operation overnight.

The radial-gate installation was subcontracted to the same company which had installed these gates on other aqueduct contracts.

Water first flowed into the Aqueduct downstream of Buena Vista Pumping Plant on September 17, 1970.

Wheeler Ridge Pumping Plant to A. D. Edmonston Pumping Plant

Design. This last 15-mile contract reach in the South San Joaquin Division extends from Wheeler Ridge Pumping Plant along the eastern edge of low hills formed by the Wheeler Ridge anticline for 2 miles to Wind Gap Pumping Plant on the eastern slope of the anticline. The discharge lines of Wind Gap Pumping Plant cross the Wheeler Ridge anticline through a topographic gap. The flow in the Aqueduct is by gravity between the Wind Gap Pumping Plant outlet and the end of the Division at A. D. Edmonston Pumping Plant.

The aqueduct alignment gradually swings southeastward through El Rincon Valley to the foothills of the Tehachapi Mountains, where it continues along the northern slope of the Tehachapis to the division terminus. The Aqueduct crosses the four drainage courses from the Tehachapis of Salt, Tecuya, Grapevine, and Pastoria Creeks.

The siphon outlet structure from Wheeler Ridge Pumping Plant and the siphon outlet structure and transition section for Wind Gap Pumping Plant were included in this construction contract; however, they are described in Volume IV of this bulletin.

Alluvial fans in this reach contain a higher proportion of coarse-grained soils with many cobbles and boulders heterogeneously mixed into the fans. For these conditions, the design specified overexcavation and backfill with an 0.8-foot blanket of select material for about 80% of the reach in order to provide a trimmable surface upon which to place the concrete lining. About 1,000 feet of potentially expansive clay encountered south of Wind Gap Pumping Plant was removed and replaced with compacted embankment.

In El Rincon Valley, areas which had been preconsolidated received special treatment. The natural ground surface under the compacted embankments was scarified to a depth of 12 inches and compacted with a 50-ton pneumatic-tired roller prior to embankment construction.

The underdrain systems of the Aqueduct between Wheeler Ridge and Wind Gap Pumping Plants and the system downstream from Wind Gap Pumping Plant serve a dual purpose, that is, not only to intercept ground water but to maintain water behind the lining below preselected levels to compensate for fluctuating elevations of aqueduct levels. Under certain operating conditions, mismatches between the three Pumping Plants in this division may occur. Therefore, the underdrain systems near these plants contain permanent, automatic, operating pumps or are easily accessible for installation of portable pumps. Pumping can lower the ground water below normal levels, 4.5, 4, and 9 feet respectively, for the downstream reaches from Buena Vista, Wheeler Ridge, and Wind Gap Pumping Plants.

The underdrain systems in this contract reach are similar to those described previously for the section below the Buena Vista Pumping Plant, except the drains are in the upper 10.5 and 13.5 feet of the lining for the Wheeler Ridge and Wind Gap sections, respectively.

In addition to the lined canal and underdrains, the main features of this contract reach consist of 3 siphons, 2 with upstream check structures and all with operating road bridges; 1 bridge to Wind Gap Pumping Plant, 1 county road bridge crossing and an oil-pipeline access bridge; 1 separate check structure; 14 overchutes ranging in size from 5 feet by 5 feet to 8 feet by 42 feet in cross section; 6 reinforced-concrete pipe culverts from 36 inches to 90 inches in inside diameter; 1 sediment trap; and 1 canal drain into Pastoria Creek. The design of these features, except the canal drain, is similar to that used elsewhere in the Division.

The canal drain, just upstream of Pastoria Creek Siphon and just downstream of the sediment trap, was designed to be used only during extreme emergency. The drain consists of an inlet structure at invert elevation which feeds a 36-inch-diameter reinforced-concrete pipe running under the canal embankment and primary operating road to an impact energy dissipator 140 feet away. From the dissipator, a 10-foot-bottom-width outlet channel with 2:1 side slopes runs to the downstream stone-protected portion of Pastoria Creek Siphon. An earth dike in the outlet channel prevents backflow from Pastoria Creek.

The drain inlet is controlled by a cast-iron, bronze-seated, sluice gate. The gate is operated by means of a hand crank located in a reinforced-concrete operating well at the top of the canal lining. The crank is linked to the sluice gate by an operating stem extending under the lining.

There was one complete turnout in this section and five single-barrel inlet structures with the remainder of the installations to be completed at a later date by the water user. There also was a three-bay pump inlet structure without gates but with individual stoplogs for each bay. Pump installation is to be completed by the water user. The complete turnout was a 30-inch pipe with hydraulically controlled gate, draft tube, control house, and equipment. The other pipe inlet turnouts were: one 30-inch, one 36-inch, one 48-inch, and two 54-inch-diameter, reinforced-concrete, pipe structures.

Construction. The contractor for this reach also had the contract for the reach immediately to the north. Since a large portion of the calendar time for the two contracts was concurrent, the work methods, procedures, and equipment generally were similar to that of the adjacent contract between Buena Vista Pumping Plant and the intake channel to Wheeler Ridge Pumping Plant.

Excavation in this reach was, however, somewhat different because of the cobbles and boulders prevalent along the alignment, all of which required overexcavation. Also, volcanic outcrops of the Tecuya formation in the hills east of Pastoria Creek required heavy ripping, although blasting was unnecessary except for an occasional large boulder. Where overexcavation in rocky areas occurred, this was done to a depth of 6 inches with replacement of compacted embankment to a depth of 8 inches. This procedure allowed the trimmer to remove the top 2 inches of material and provide a good subgrade for the lining.

A small slide occurred in the Tecuya formation about ½ of a mile upstream of Pastoria Creek Siphon during canal excavation in January 1969. The interbedded layers were inclined toward the excavation and intersected by a steeply dipping fault. The slide occurred shortly after the area had been soaked by heavy rains. The cut was resloped at 2¾:1, essentially removing the failure plane. The canal prism was overexcavated to 3 feet below invert elevation and backfilled with compacted select material.

The forms for structural concrete generally were interchangeable for the siphons and check structures; however, the relatively steep slopes at Grapevine Siphon required the use of a slip form paver on the invert and top slab, and retaining forms for the walls. H-beam pilings were used for seven of the bridges with footings for the remaining bridges, overchutes, and pipeline crossings.

The shape of the collector trench for the finger drains was changed from triangular to trapezoidal to reduce the span of the lining across the trench and thus reduce the cracking potential if good compaction was not obtained for the filter material.

Two inaccessible areas under an overchute and under the Interstate 5 overcrossing were lined with shotcrete. This method worked quite satisfactorily. Shotcrete also was used to line some of the drainage ditches adjacent to the canal section.

Initial Operations

Measuring wells, survey nets, and photographs of as-built conditions were used to supplement visual information during initial operations. The actual operation of the conveyance facilities through the areas of hydrocompactive soils was attentively watched. With ground water a lesser problem than in the North San Joaquin Division and with the majority of the Aqueduct in cut sections, less settlement and lining cracking was expected in this division. This assumption proved to be accurate.

Kettleman City to Avenal Gap

Avenal Gap Siphon has counterfort walls in both the intake and outlet transition sections. Rather large separations of the earth from the walls occurred at the counterfort section in both upstream and downstream walls. Proper backfill compaction is difficult and troublesome to achieve against these counterforts, and the pulling away or settlement of the compacted backfill adjacent to counterfort walls became a common occurrence for the conveyance facilities.

For Avenal Gap Siphon, a mixture of sand and earth was injected into the backfill area using 1½-inch pipes. Where the ground had cracked, Bentonite slurry was injected into the larger cracks. Additional settlement has been minimal.

Downstream of the Interstate 5 crossing, subsidence occurred at the primary operating road where a 60-inch-inside-diameter culvert crosses under the road. The subsidence spread to the west side of the canal with some cracking of the lining. A series of piezometer wells were established to monitor the area. No moisture was detected and, after refill of the initial subsidence, the ground was stable. Scuba divers investigating the underwater conditions filled a series of lateral cracks near the bottom of the canal with All-crete patching material and a sealant.

Avenal Gap to 7th Standard Road

No problems of a serious nature occurred in this reach. Normal maintenance took care of such problems as better drainage along spoil banks or minor ponding at drainage structures.

7th Standard Road to Tupman Road

This reach also had only minor problems. Downstream of Temblor Creek Siphon, some settlement and cracking of the ground adjacent to the east side of the Aqueduct occurred. Underwater investigation of the lining disclosed no damage to the lining of the canal.

Tupman Road to Buena Vista Pumping Plant

There were only minor drainage problems in this section. The ground water relief system for the localized water table upstream of Buena Vista Pumping Plant worked satisfactorily.

Buena Vista Pumping Plant to Wheeler Ridge Pumping Plant

Settlement over a 72-inch-inside-diameter concrete culvert occurred just downstream of Buena Vista Pumping Plant. Settlement of the operating road and cracking of the adjacent canal lining were similar to the occurrence near the Interstate 5 crossing to the north. The subsidence occurred some time after the canal was filled and apparently was a slow progressive-type settlement. The lateral cracks in the lining were patched with Allcrete and no further cracking or settlement has occurred.

Wheeler Ridge Pumping Plant to A. D. Edmonston Pumping Plant

This reach, too, had very minor problems. The sublining or overexcavation and backfill for rocky ground which were done for much of this section apparently resulted in a very stable foundation for the Aqueduct.

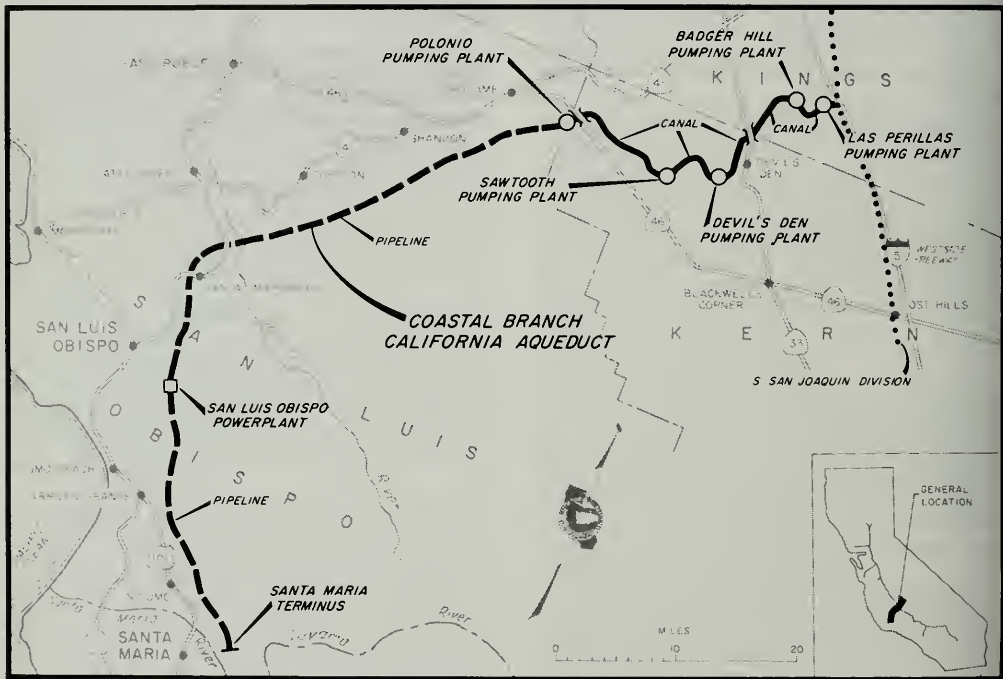

Figure 173. Location Map—Coastal Branch

CHAPTER VII. COASTAL BRANCH

Introduction

Role in the State Water Project

The Coastal Branch, when fully completed, will extend about 96 miles from the main California Aqueduct at Milepost 184.63, near Kettleman City in the San Joaquin Valley, to a terminus at the Santa Maria River at the San Luis Obispo-Santa Barbara county lines (Figure 173). Phase I, the first 15 miles, was placed in operation in January 1968 in Kings and Kern Counties (Figure 174). It consists of some 14 miles of canal, Las Perillas and Badger Hill Pumping Plants and discharge lines, with the terminus at Berrenda Mesa Water District's pumping plant (near the site of the future Devil's Den Pumping Plant planned under Phase II). See Volume IV of this bulletin for a description of Las Perillas and Badger Hill Pumping Plants. Water deliveries are made to Devil's Den Water District with a maximum annual entitlement of 12,700 acre-feet and to Berrenda Mesa Water District, a member unit of the Kern County Water Agency, which Agency has a maximum annual entitlement of 155,100 acre-feet from the Coastal Branch. Under Phase II, present plans are that the remaining 81 miles of the Coastal Branch will be completed in about 1982, delivering water to San Luis Obispo County Flood Control and Water Conservation District and Santa Barbara County Flood Control and Water Conservation District, with maximum annual entitlements of 25,000 and 57,700 acre-feet, respectively. This phase will include three additional pumping plants and a power-recovery plant, with several miles of canal and pipelines.

Hydraulic Function

Flow in Phase I facilities of the Coastal Branch is conveyed through canal section and pipeline and is lifted at Las Perillas and Badger Hill Pumping Plants through a total static head of about 206 feet. Flow is controlled by gated check structures which, along with the two pumping plants, divide the Aqueduct into six pools operated under the "controlled volume concept" discussed earlier in this volume and in Volume V of this bulletin. A statistical summary of Coastal Branch conveyance facilities is presented in Table 14.

TABLE 14. Statistical Summary of Coastal Branch

CANAL

Type
Concrete-lined—trapezoidal—checked

Dimensions
Lined depth, 6.96 feet; bottom width, 8 feet; side slopes, 2:1; length, 14.2 miles

Capacity
450 cubic feet per second

Freeboard
1.5 feet lined and a minimum of 2.0 feet of earth berm above lining

Lining
3½-inch unreinforced concrete—sealed longitudinal and transverse contraction joints on a maximum of 7- and 10-foot centers, respectively

Bridges
2 vehicular—2 farm

Check Structures
2 two-radial-gate structures

Cross-Drainage Structures
47 culverts—26 overchutes—15 drain inlets

OPERATIONS
Manual on-site control or remote control from area control center, San Joaquin Field Division

Geography, Topography, and Climate

The Coastal Branch in south central Kings County and northwestern Kern County crosses the Kettleman-Antelope Plain, passes the southern end of Pyramid Hills, and terminates at Berrenda Mesa's pumping plant at the base of the foothills of the Coast Range. The area traversed is agricultural, with a mild winter climate and long, hot, dry summers. Temperatures range from a low in the mid-20s during the winter to a high slightly above 100 degrees Fahrenheit during the summer months. Rainfall increases as the Coastal Branch moves away from the San Joaquin Valley; it averages about 12 inches per year and occurs primarily during the winter months. Local floodflows result both from general rainstorms and from isolated thunderstorms. General storm runoff is characterized by heavy peak flows, moderate flood volume, and durations of from one to two days.

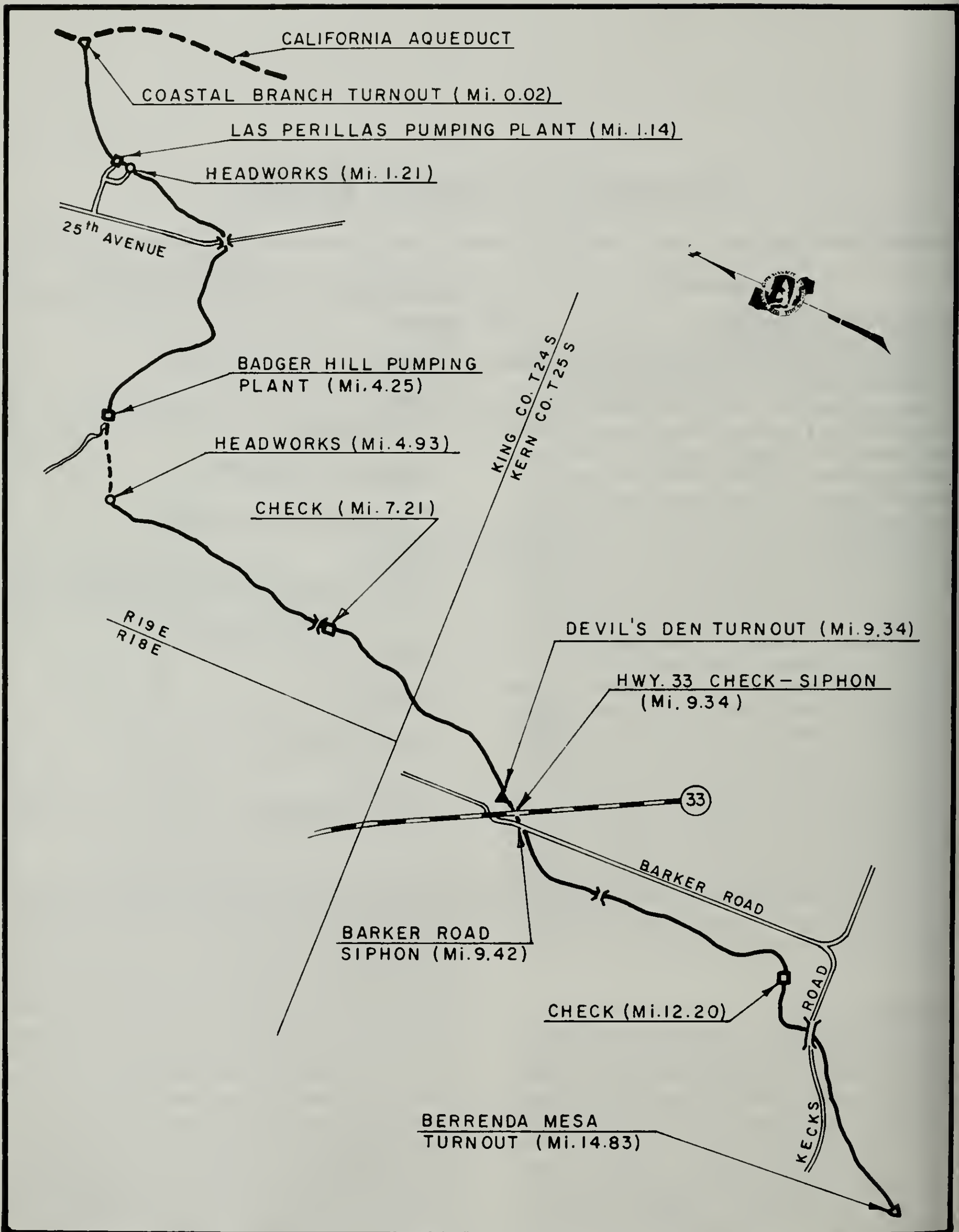

Figure 174. General Plan of Coastal Branch—Phase I

Geology and Soils

Geology

The Coastal Branch aqueduct in the San Joaquin Valley traverses the eastern foothills of the Coast Ranges in an area of varied geologic structure. After branching westward from the California Aqueduct, the branch aqueduct passes through Avenal Gap, a small valley between the middle and south domes of Kettleman Hills. Kettleman Hills comprise a northwest-trending anticlinal structure whose surface expression consists of three low domes of Tertiary sedimentary rocks which protrude through the alluvium of the San Joaquin Valley floor, referred to as North, Middle, and South Domes. Going through Avenal Gap, the canal is located on the north side of the Middle Dome and encounters primarily poorly consolidated, interbedded sands and clays of the Pliocene San Joaquin formation. Some alluvial deposits also are encountered in the Avenal Gap area along with poorly consolidated sands, gravels, silts, and clays of the Plio-Pleistocene Tulare formation.

Leaving Avenal Gap, the alignment skirts the western edge of Kettleman Hills and then crosses the Kettleman-Antelope Plain, which is a syncline filled with alluvial deposits consisting of clays, silty sands, and silty gravels. Going west from Kettleman-Antelope Plain, the conveyance system skirts the southern edge of Pyramid Hills. In this location, some fractured siliceous shale and hard sandstones are present in short reaches, but mostly the canal is in alluvium consisting of poorly consolidated sands and clays. Tertiary sedimentary rocks throughout much of the Pyramid Hills are complexly folded and faulted but, in the vicinity of the canal alignment, primarily are northeasterly dipping beds on the flank of an anticline.

Some minor faulting is present in this area, particularly in the vicinity of Kettleman Hills. This faulting was developed during the domal uplift of the sedimentary rocks. No major faults are crossed, and the nearest major fault zone is the San Andreas, about 10 miles to the west.

Soils

Soils excavated consisted of clays, silts, silty sands, and gravels. In addition to material excavated from the canal, additional borrow was developed in the Kettleman-Antelope Plain to supply material for the fill section across this area.

Gypsum-rich soils are prevalent in the region, and several areas nearby are commercially mined to obtain gypsite for agricultural purposes. Where gypsum-rich soils were encountered in the canal excavation, they were overexcavated and replaced with compacted sublining. Potentially expansive clays and low-strength soils were treated in the same manner. Some areas of excavation, through the harder Tertiary rocks, were overexcavated and replaced with compacted sublining so that suitable trimming could be done.

It was suspected that spotty shallow subsidence might be encountered along the alignment. Approximately 13,000 feet of alignment was estimated to require surface application of water for preconsolidation.

In the area of deep cuts, there is high ground water of extremely poor quality. Since this water is injurious to crops and livestock, it is pumped into large evaporation ponds when disposal becomes necessary.

Design

The Coastal Branch facilities for Phase I were designed to be constructed under two contracts. Design of the aqueduct system provides for a controlled gravity system with a continuous 24-hour operation. Flow is predicated upon the downstream demand. Complete monitoring of all aspects of this system is maintained so that the nature of operation throughout is known in the area control center which can command the operation of all control facilities. If canal failure occurs without warning, pumps will be shut off and check gates closed to keep both damage and loss of water to a minimum. Local automatic and override operation also was provided at all control structures. The Aqueduct was designed to be operated in such a manner that it will be full of water during normal operation. The 1.2-mile reach of canal from the California Aqueduct to Las Perillas Pumping Plant operates with the same water surface elevation as the California Aqueduct. The 3.7-mile canal reach between Las Perillas and Badger Hill Pumping Plants has enough storage to contain about 45 minutes of maximum flow before overtopping. The 9.2-mile reach from Badger Hill Pumping Plant to the terminus can contain about 50 minutes of maximum flow. With the automatic alarm system in operation and coordinated operation with Berrenda Mesa's pumping plant, this provides adequate time for adjustments at the pumping plants, and no emergency spill facilities were necessary. A full emergency shutdown of either Las Perillas or Badger Hill Pumping Plants may cause a surge of approximately 2 feet in the canal. In view of the infrequency of such occurrences, this can be tolerated.

Hydraulic Structures

In general, concrete design of the canal and hydraulic structures was in accordance with ACI 318-63 and ACI 315-65. Steel reinforcement was specified as intermediate-grade or hard-grade billet steel in accordance with ASTM Designation A15. Design of structural steel was based on "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings" of the American Institute of Steel Construction. Design of fabricated aluminum was based on the "Alcoa Structural Handbook" of the Aluminum Company of America and the "Aluminum Construction Manual" of the Aluminum Association. Aluminum alloy 5456-0 was required because of its combination of weldability and resistance to corrosion with relatively high strength.

Bridges

Four reinforced-concrete bridges were designed and constructed in the Coastal Branch (Figure 175). These bridges were built to (1) replace sections of the existing roads which were severed by the Aqueduct, (2) provide access across the Aqueduct for property owners, and (3) provide access across the Aqueduct for operations and maintenance vehicles. Design of the bridges was based on AASHO "Standard Specification for Highway Bridges" and on the California Division of Highways' (now the Department of Transportation) "Bridge Planning and Design Manual". All bridges were designed to the AASHO loading of H20-S16 and to the Bureau of Public Roads alternative loading of two 24-kip axles at 4-foot spacing. In addition to the above design loads, the farm bridge at Milepost 10.45 was designed to carry the weight of one large tractor and truck-tractor and (low-boy) trailer with a gross weight of 110 kips. This bridge has 4- by 12-inch planking on the deck so that it can be crossed with nonrubber-tired equipment.

Embankment and Cut Slope

Soil and geology reports delineated reaches where soil problems might exist and recommended measures for dealing with the problems. Recommended soil-strength values for slope stability were based on the Swedish Slip Circle method of analysis. The conditions analyzed for slope stability were: operation with 0.1g seismic loading, construction with 0.1g seismic loading, and drawdown. The phreatic surface for operational and drawdown conditions was considered to be at the normal water depth extending horizontally through the section and for the construction condition was considered to be at the ground water table.

Canal Excavation and Embankment

Slope stability analysis indicated that the side slopes in most reaches should be set at 2:1 to achieve a factor of safety of 1.2. However, this factor of safety cannot be realized for operating conditions in reaches where the recommended value of cohesion is zero. The assumed conditions of the phreatic surface location and zero cohesion made for a too conservative approach; therefore, a factor of safety of 1 was considered to be tolerable under these assumed conditions.

Design of the embankments was based on information from the geological exploration program and laboratory testing of selected samples. The embankment was designed in anticipation of foundation settlement due to clayey soil foundation. Compacted embankment material was selected on the basis of the following qualities: workability, compactibility, adequate shearing strength, and low permeability. In fill sections, the compacted embankment has a 6-foot width at the top of the lining and side slopes of 2:1 in the canal prism and 1:1 within the embankment (Figures 176 and 177). The remainder of the fill sections are comprised of uncompacted materials on which the roadways were constructed. Waste materials were spilled on the outside of the roadways or in designated spoil areas.

Figure 175. Canal Road Crossing

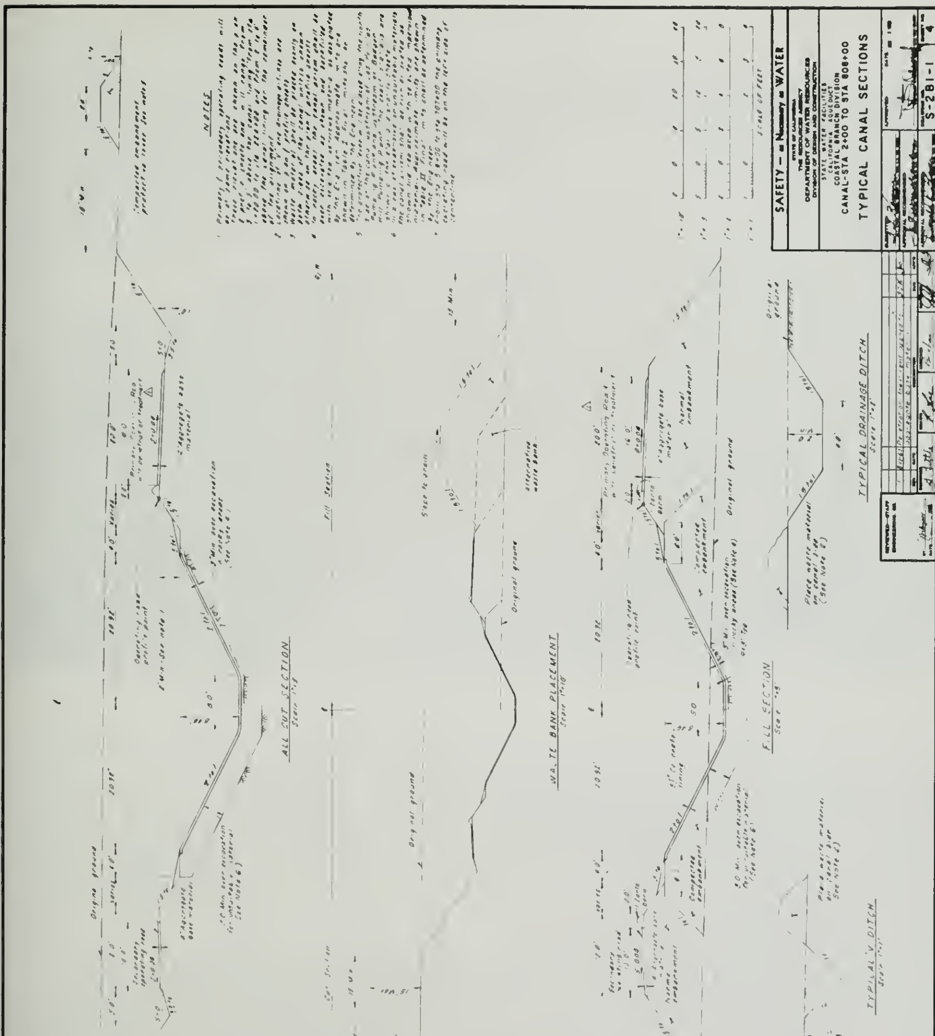

Figure 176. Typical Canal Sections

Figure 177. Coastal Branch Canal

In areas where the original ground is below canal invert, the compacted embankment was completed to full height at least 90 days prior to trimming and lining. Embankments upon which abutments and footings were constructed were completed at least 60 days prior to placement of concrete to allow for settlement of foundation material.

The final alignment was such that, generally, the lining is not in fill, and the mass diagram had a slight positive slope so that adequate quantities of suitable materials were available for construction of embankments. The minimum curve radius is five times the top width of the canal lining.

The alignment between Badger Hill Pumping Plant and the present terminus of the canal was selected to ensure that the canal invert would not be in fill. Using this point as a pivot, rough earthwork quantities were computed for various slopes. The slope selected is one which minimizes the earthwork and keeps the lining in the original ground. The same slope was fitted to the ground between Las Perillas and Badger Hill Pumping Plants for various headworks elevations, and rough earthwork quantities were computed for each elevation. The selected elevation minimizes the earthwork between the two pumping plants.

An underdrain system was installed wherever the ground water table was found to be within 1 foot of canal invert to allow for the possibility that the water table might rise in future years. The underdrain system consists of a 6-inch filter blanket under the entire canal prism; an 8-inch-diameter, perforated, collector pipe; an 8-inch header pipe which conveys the drain water to sumps; an 8-inch riser pipe at each end of the system to allow for periodic flushings; and evaporation ponds for disposal. Since the ground water is of extremely poor quality, evaporation ponds were constructed for disposal. The ponds utilize the top of the waste banks of the pumping plant excavation. The underdrain system was designed for both asbestos cement and clay underdrain pipe.

Measuring wells were placed at intervals along the Aqueduct to provide a means for determining the ground water level. They consist of a 15-foot length of 2-inch-diameter polyvinyl chloride pipe perforated at the base and placed in a 6-inch-diameter drilled hole that was backfilled with peagravel. They are located immediately adjacent to the canal lining on the primary operating road side of the canal.

Coastal Branch Turnout

The turnout (Figure 178) consists of an inlet structure, two rectangular boxes under the operating road, two rectangular bays equipped with stoplog grooves, an outlet transition, and a control building. The first reach of the Coastal Branch operates at the same water surface elevation as that of the California Aqueduct. The outlet transition is identical to the outlets on the checks, check-siphon, and siphon, except for the invert slope which is zero and the reinforcing steel, and reflects the soil conditions at the site.

Headworks

The headworks structures (Figure 179) at the end of the pumping plant discharge lines consist of two cast-in-place, round-to-square, concrete transitions; two 7-foot-wide rectangular bays with radial gates and stoplog grooves; and an outlet transition. It was believed that savings could be realized if both headworks structures were identical. Therefore, the design was based upon the maximum loading condition at either site. The design loading conditions used for analysis were saturated embankments, H20-S16-44 surcharge, and maximum lateral earth pressure for the following conditions: (1) one bay full, (2) structure empty, and (3) structure full. The one-bay-full condition applies only to bay design. The bays were designed to accommodate the radial gates, to be as nearly identical as possible to the other checks, and to withstand the lateral earth pressures recommended for the site in the soils report. The outlet transition has a level invert and a nose insert which was designed to produce a smooth reduction in velocity through the transition, thus minimizing head loss. The walls diverge at 22½ degrees, which is an angle recommended for diverging transitions based upon model studies of inlet and outlet transitions for the California Aqueduct. The radial gates are the same basic gates as those used on the checks and the check-siphon. The face plate above the upper arm was shortened because of a lower water depth at the headworks sites. The purpose of the gates is to prevent draining of the canal through the discharge lines should the pumps fail. Stoplog grooves were provided at the downstream end of the bays.

Figure 178. Coastal Branch Turnout From the California Aqueduct

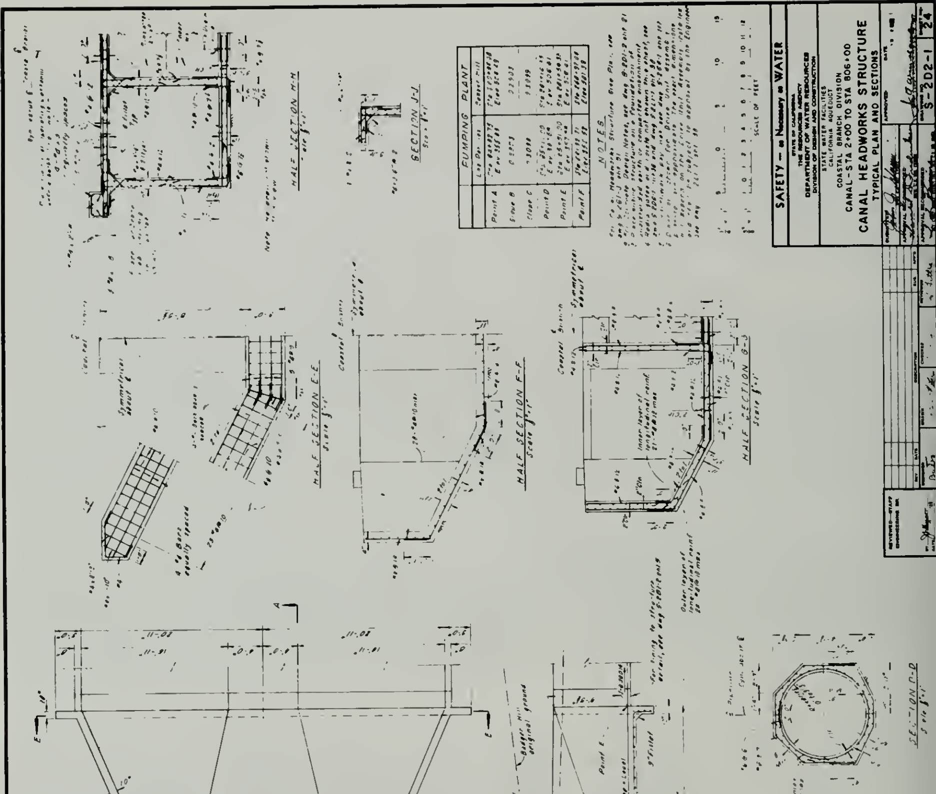

Figure 179. Canal Headworks Structure—Plan and Sections

Checks

The checks (Figures 180 and 181) are cast-in-place concrete structures consisting of inlet and outlet transitions, two 7-foot-wide rectangular bays equipped with motor-driven radial gates and stoplog grooves, and a control building containing an auxiliary power supply and monitoring equipment. Functional requirements of the checks are to:

1. Maintain a minimum water surface elevation for turnouts and pumps.
2. Prevent damage to canal lining caused by a combination of a high ground water table and large fluctuations in the canal water surface.
3. Isolate any reach for repair or maintenance.
4. Contain the in-transit flow within the reach when a shutdown occurs.

The approximate maximum check spacing was determined by using the criteria that: (1) static water surface may be allowed to encroach on 75% of the lined freeboard with a minimum of 0.3 of a foot of freeboard being maintained, (2) a minimum water depth of two-thirds normal depth shall be maintained, and (3) maximum spacing of checks shall be 16,000 feet.

A 7-foot-wide bay was selected based on an economic study which considered the cost of the structure and gates and the cost of the attendant head loss. The Coastal Branch turnout, headworks structure, checks, and check-siphon all have 7-foot bays and all, except the turnout, have radial gates. The inlet and outlet transitions used on the Coastal Branch turnout, checks, check-siphon, and siphon are identical except for the invert slope and the steel reinforcement. Hydraulic design was based upon model studies of inlet and outlet transitions for use on the California Aqueduct. The inlet walls converge at 30 degrees and the outlet walls diverge at 22½ degrees. The slope of the invert of the check inlet is zero. The invert slopes of the check outlets are −0.0036 and −0.0027. The check outlets were sloped to recover losses through the checks, and slopes vary because of different design flows.

The radial gates (Figure 182) are remotely controlled with manual overrides and are operated from an adjacent control building and the area control center. Design of the radial gates was based on: (1) design stresses increased by 33% for abnormal conditions; (2) use of A36 steel; (3) a factor of safety of 8 for anchor bolts; (4) gate arms load increased 25% of direct load for a friction moment; (5) radius on inside of face plate to equal 1.25 (H-1) where H is the maximum height for the gate in feet; (6) pin height between 0.5 (H-1) and 0.75H; and (7) distance along inside of the face-plate arc from the sill to the centerline of the lower arm equal to 0.123L and from the centerline of the lower arm to the centerline of the upper arm equal to 0.491L, where L is the arc length of the face plate. Gates were not designed for partial static pressure and dynamic pressure from underflow because static pressure will be a more critical condition.

The loading conditions for the gates were analyzed as follows:

1. Upstream water depth of 8.23 feet, downstream empty.
2. Downstream water depth of 6.33 feet, upstream empty.
3. Overflow water depth of 10.60 feet, downstream empty.

The check-siphon on State Highway 33 has inlet and outlet transitions; two 7-foot-square, cast-in-place, concrete barrels; two check bays; transitions between the checks, bays, and square barrels; transitions between the barrels and access bays; and two access bays. Highway 33 is crossed by a siphon because a cross-drainage problem exists at this location and a bridge alternative compared unfavorably with the siphon. The siphon was designed for a capacity of 450 cubic feet per second (cfs). Inlet and outlet transitions are identical to the inlet and outlet transitions of the checks. The inlet is sloped to (1) maintain a positive water seal, (2) minimize bend loss in the barrels, and (3) shorten the siphon. The two 7-foot-square, cast-in-place, concrete barrels were designed for lateral earth pressure; saturated conditions and H20-S16-44 live loads; full external load, no internal load; full internal load, no external load; one box full, no external load; one box full, full external load; and full external load, full internal load. The siphon barrels were extended an additional length on both sides of Highway 33 to allow the owner of the adjoining land access across the Aqueduct without using the Highway and to provide a channel to pass the drainage across the Aqueduct.

A siphon at Barker Road was added by a change order. The Barker Road siphon consists of an inlet and outlet transition; stoplog grooves; two 7-foot-square, cast-in-place, concrete barrels; a transition from the barrels to the access bays; and two access bays with stoplog grooves. The siphon was designed to be nearly identical to the Highway 33 siphon. Two water lines have been relocated in this area: a 36-inch, precast, concrete, irrigation line under the siphon and a 2-inch steel water line under the siphon.

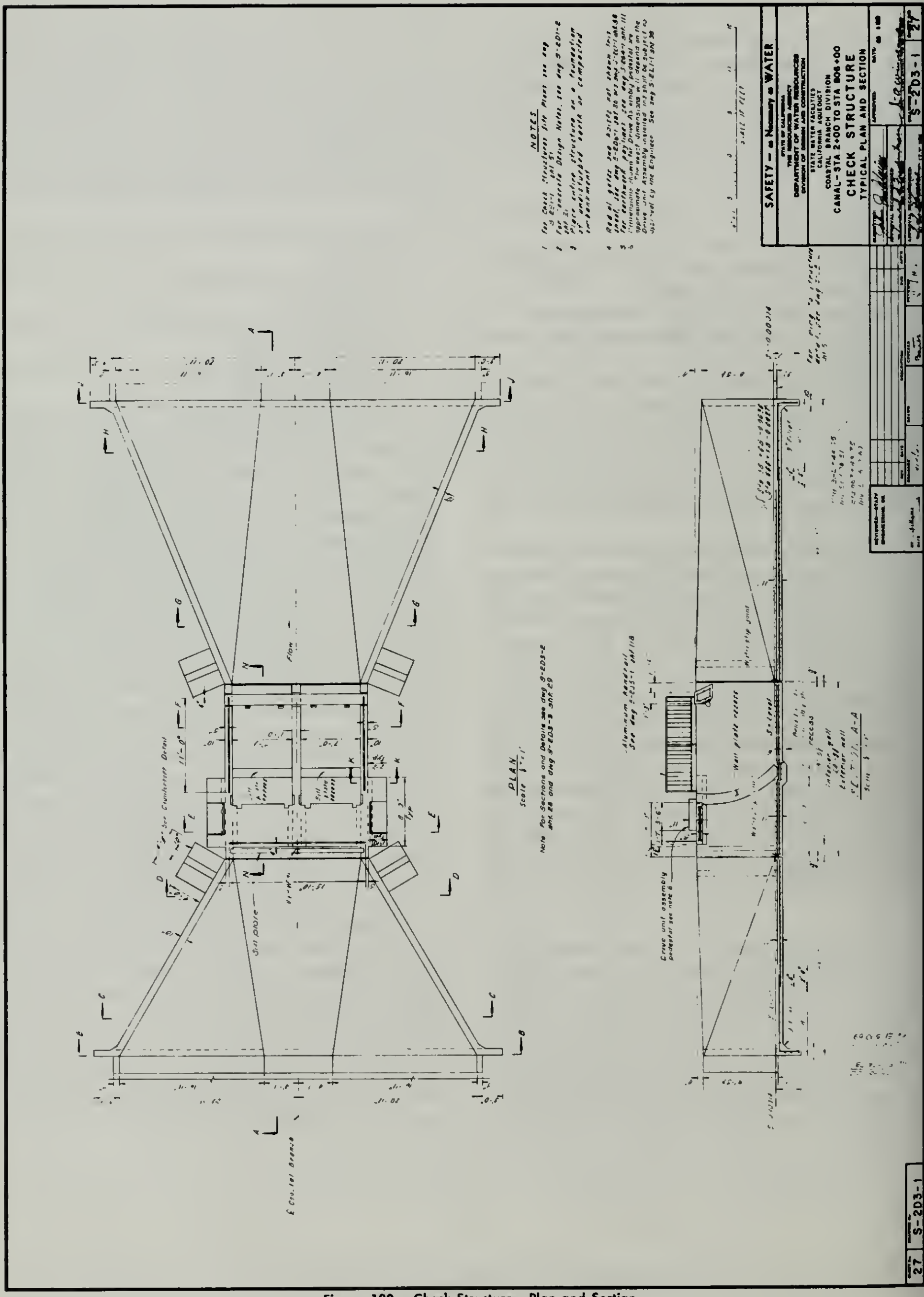

Figure 180. Check Structure—Plan and Section

Figure 181. Completed Check Structure

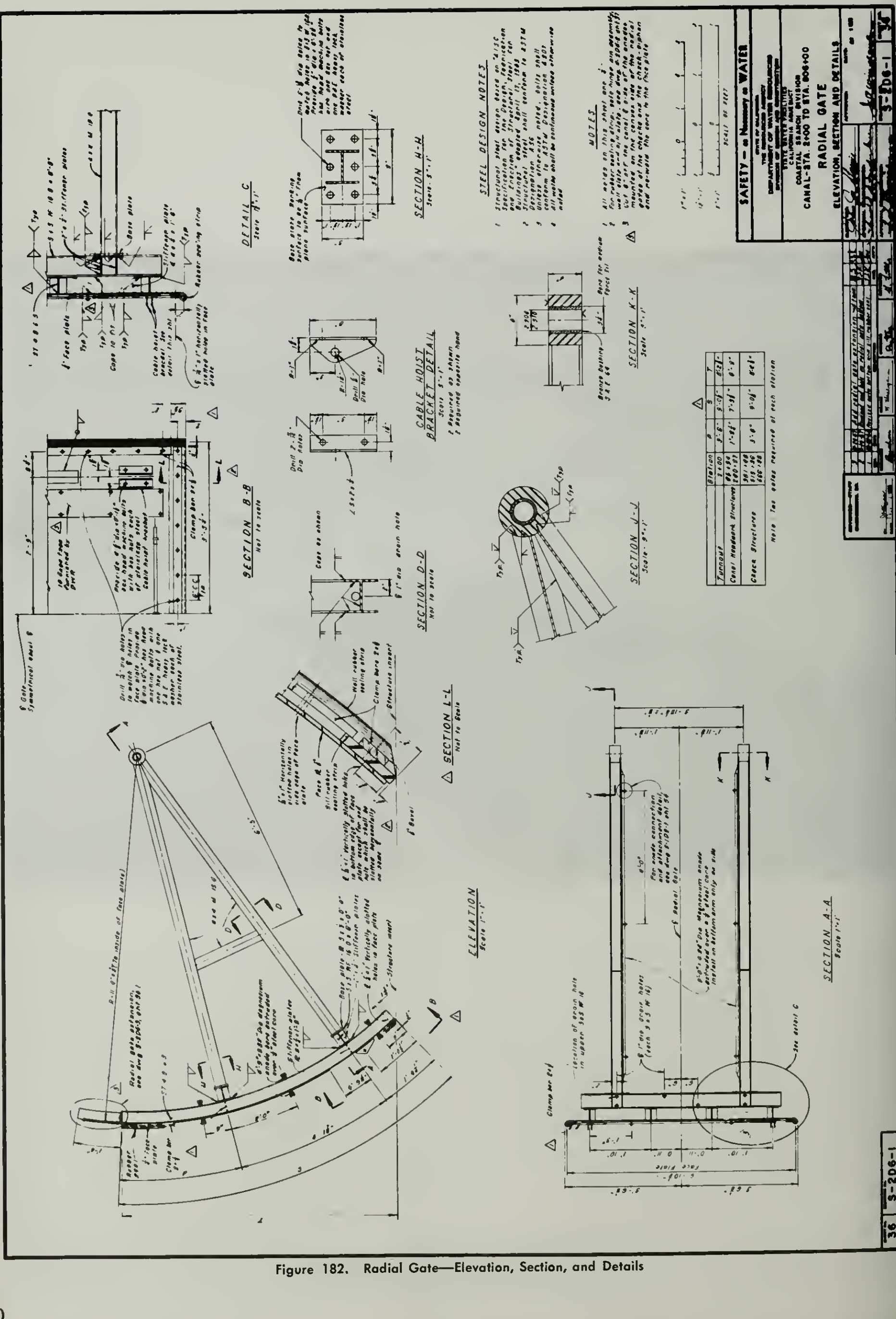

Figure 182. Radial Gate—Elevation, Section, and Details

Turnouts

Devil's Den turnout at Highway 33 consists of a remotely controlled hydraulic slide gate; a flow tube; a control building; and 91 feet of 54-inch, precast, concrete pipe. The turnout is operated from an adjacent control building by the water user. The location, design discharge, and delivery water surface were determined by the water user, and the turnout barrel was sized to comply with these conditions. The slide gate is used to regulate the flow. Functional criteria specified for the turnout were:

1. The turnout be provided with equipment to control and measure water deliveries.
2. A differential head measuring device be provided with an accuracy of plus or minus 2%.
3. The turnout have facilities for remote flow control.
4. The rate for varying flows through the turnout be small enough to eliminate operational problems in the canal.
5. Provision for calibrating the flowmeter be incorporated.
6. The flowmeter be submerged at all times.
7. Trashracks be provided at the inlet structure on a slope of 1:1.

The maximum design flow is 65 cfs. Devil's Den Water District has a contract for water delivery at a peak rate of 38 cfs but requested that a 65-cfs-capacity turnout be provided to enable them to take delivery of additional water through this turnout that may be purchased from Berrenda Mesa Water District.

Water is delivered to Berrenda Mesa Water District at the canal terminus by extending the canal section to the intake channel of Berrenda Mesa's pumping plant. Flowmeters were installed in the discharge lines, and the Department of Water Resources monitors the flow by remote control facilities placed in the control building for the pump turnout. The intake for Devil's Den Pumping Plant will be placed in the side of the canal in the completion phase of this branch.

Stoplogs

One concrete and three steel stoplogs were provided for use in canal structures. Two of the steel stoplogs are identical and allow maintenance on the discharge lines, siphon barrels, and radial gates while the canal is in service. The third steel stoplog permits Devil's Den turnout to be hydraulically isolated from the canal. The concrete stoplog was added when it was decided to stage the construction of the Badger Hill Pumping Plant discharge lines. The semipermanent concrete stoplog was used to seal the pipeline stub until the second discharge line was completed.

Sand Traps

Several reaches of the canal are affected by wind-blown sand. Problem areas are the canal reaches between the Coastal Branch turnout and Badger Hill Pumping Plant and between Milepost 10.5 and Berrenda Mesa turnout. Sand traps were provided to catch the bed load. There are five sand traps, located as follows: immediately upstream of Las Perillas and Badger Hill Pumping Plants, immediately upstream of the two turnouts, and immediately downstream of Milepost 12.96. The sand traps were located on tangents with a long reach of tangent upstream where possible. Each trap consists of a reinforced-concrete structure below canal invert. Each has two consecutive chambers, a hinged aluminum grating in each chamber, and a hand-winch lifting assembly for each grating. Each chamber is 10 feet long, 4 feet deep, and 8 feet wide. The aluminum gratings are formed by a rectangular plate and channels attached to a frame. Accumulated sediment can be pumped out with a suction hose while the canal is operating. The sand traps usually are cleaned when the canal is drained for other purposes.

Drainage Structures

The drainage facilities consist of drain inlets, culverts, overchutes, ditches, and levees located at drainage channels and low points along the Aqueduct. Small subbasins along the canal are ditched and filled so they will drain into adjacent basins which are drained by a cross-drainage structure. In addition to the general requirements for cross-drainage structures, the following criteria were adopted:

1. No permanent ponding of water.
2. Energy dissipators where outlet velocities exceed 12 feet per second; however, large dissipating outlet structures on all overchutes and culverts larger than 24 inches, regardless of outlet velocities.
3. Water not allowed to stand in the barrels of the cross-drainage structures.
4. All culverts under the canal of precast concrete pipe or cast-in-place concrete structures; top of the culvert a minimum of 3 feet under the invert of the canal.

Drain inlets were provided to convey the small amount of runoff into the canal from the operating roads and cut side slopes. They consist of a reinforced-concrete collector box to receive the water from the operating road ditch and a length of 8-inch-diameter corrugated-metal pipe to carry the water under the road to the canal.

Each culvert installation consists of a concrete box, a pipe extending beneath the canal and operating roads, inlet and outlet structures, and stone slope protection at the inlet and outlet.

An overchute is either a pipe crossing or a rectangular flume (Figures 183 and 184). A pipe overchute consists of steel pipe reaching across the canal supported by concrete abutments, attached lengths of corrugated pipe extending beneath the operating roads and inlet and outlet structures, and stone slope protection. Flume overchutes are concrete rectangular boxes

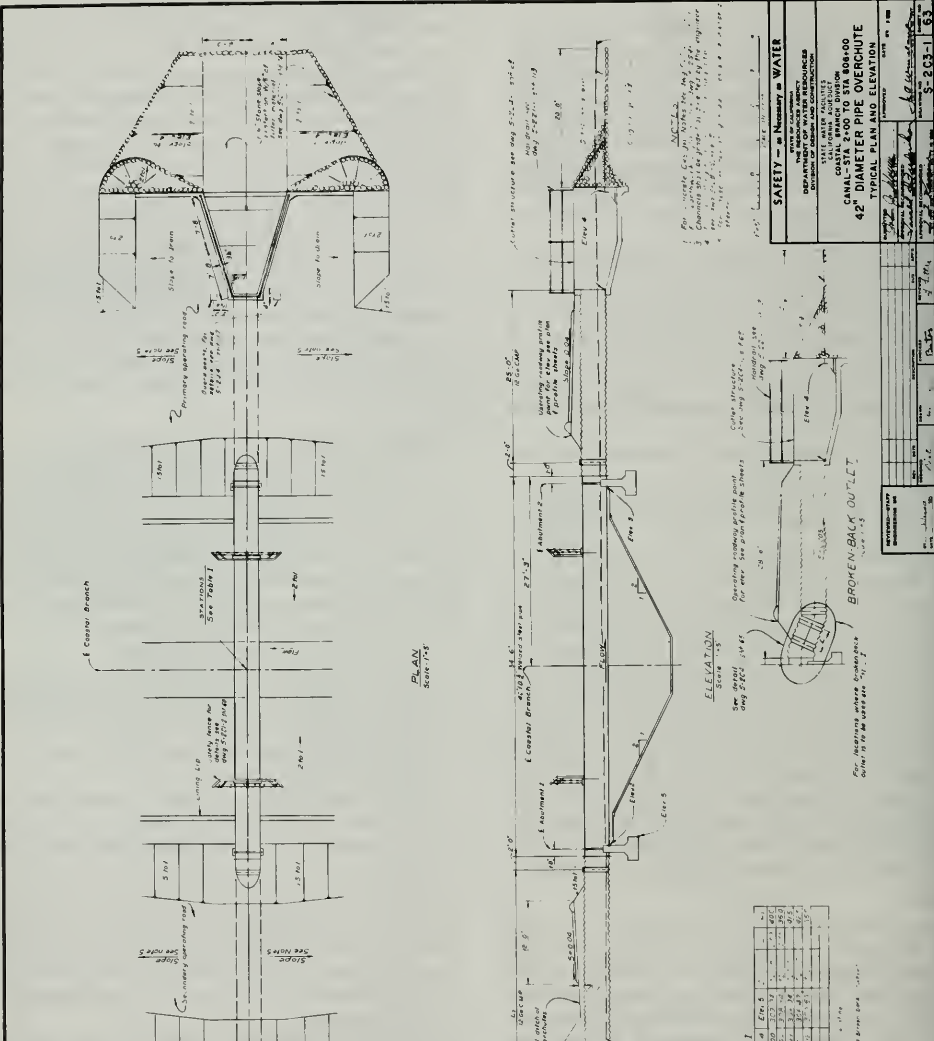

Figure 183. Pipe Overchute—Plan and Elevation

Figure 184. Completed Overchute

extending beneath the operating roads and continuing as open boxes across the canal, inlet and outlet structures, and adjacent stone slope protection.

Barker Road drainage ditch lies adjacent to the west edge of Barker Road and crosses the Barker Road siphon. The drainage ditch is a shallow earth channel which has a base width of 12 feet where it crosses the siphon and a base width of 20 feet beyond the crossing.

The levees are protective embankments near Las Perillas and Badger Hill Pumping Plants. They prevent storm runoff, flowing east toward the Avenal Gap Siphon on the California Aqueduct, from entering Las Perillas and Badger Hill Pumping Plant sites or the canal in this vicinity. The protective levees are at least 5 feet above original ground and the Las Perillas Pumping Plant protective levee is at least 5 feet above the flood stage anticipated at the California Aqueduct. The top width of the levee is 5 feet and maximum height approximately 6 feet.

The canal operating road ditches are 1 foot deep with an invert slope of 0.25% or greater. The flow from these ditches is either conveyed to drain inlets or permitted to discharge into the adjacent right of way where it is carried off by the cross-drainage facilities and flows away from the canal.

Relocations

Relocation of gas (Figure 185), oil, power, and telephone lines was accomplished by the owner utility company. The canal contractor did the site preparation by constructing the canal prism and operating roads in the area of relocation prior to relocation by the pipeline owner. The carrier pipes, abutments, and connections were designed by the pipeline owner. Design of abutments and casing was accomplished by the Department. All lines were cased across the canal and vented outside the operating roads. Relocation of water supply facilities was accomplished by the canal contractor and included siphons, overchutes, and pipelines. Design of these facilities by the Department was approved by the owner, and the facilities were provided in accordance with property settlement agreements.

Irrigation siphons were provided to connect severed irrigation canals. Design criteria required that the top of the pipe be at least 3 feet below invert of canal, using the same class and type of pipe as for drainage facilities, with the right-of-way fence placed so that the irrigator has access to the inlet and outlet. Design of the 24-inch-diameter irrigation overchutes was identical to that of the 24-inch drainage overchutes.

Roads

Operating roads are located on each side of the canal prism. The primary operating road is 20 feet wide with a surface width of 16 feet, and the secondary operating road is 12 feet wide with a surface width of 10 feet. Maximum profile grade is 7% except for ramps onto public roads, where the maximum grade is 20%. The roadway slopes away from the canal at 4%. A minimum cover of $1\frac{1}{2}$ feet was provided over all structures under the operating roads. The roads have a 4-inch gravel surfacing, and the primary operating road has an asphalt penetration treatment applied to the surfacing. A metal guardrail was provided at locations on the operating roads where aqueduct structures were adjacent to the roads.

Paved access roads were provided to Las Perillas and Badger Hill Pumping Plants and to the respective discharge line headworks structure.

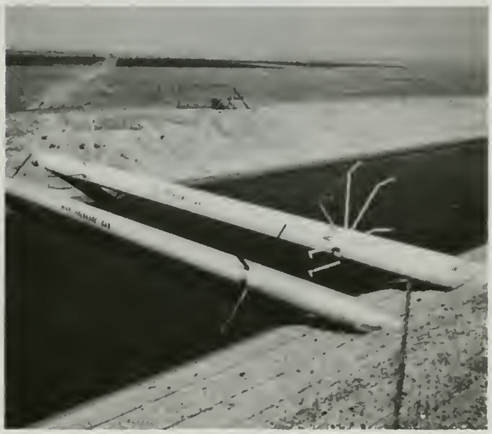

Figure 185. Gas Pipeline Crossing

Construction

Construction supervision was administered by the Bakersfield Project Office with inspection provided by the Avenal Field Office. A soils and concrete laboratory was established at Taft, California. General information about the major contracts is shown in Table 15.

Construction work commenced in January 1966 and was completed in June 1968. Inclement weather was not a factor during construction.

Preconsolidation Features

This work involved the construction of preconsolidation features in the vicinity of Devil's Den between Milepost 10.6 and Milepost 14.0. The work consisted of the construction of 13 subsidence ponds and culverts to convey water to and between the ponds.

Water for ponding was supplied by South Lake Farms under a service agreement. Midway Drilling and Pump Company delivered the water from the South Lake Farms canal to the ponds. Water deliveries began on February 28, 1966 and ended on June 15, 1966. The total quantity of water delivered was 577.55 acre-feet. Fortunately, little or no subsidence occurred.

Aqueduct Features

This work consisted of the construction of approximately 14.2 miles of canal from Avenal Gap to the proposed site of Devil's Den Pumping Plant. In addition to the canal proper, the work consisted of numerous utility relocations, road detours, bridges and other miscellaneous concrete structures, evaporation ponds, access roads, and first-stage excavation for both Las Perillas and Badger Hill Pumping Plants.

Excavation for all construction was performed by conventional methods using dozers, scrapers, and backhoes. Construction of the canal involved excavation of the canal prism, overexcavation in areas of unsuitable material, compacted embankment within the canal prism, and placement of canal embankment. Gypsum deposits were encountered throughout the alignment. Any concentrated deposit of gypsum encountered during construction was overexcavated and backfilled with a compacted layer of impervious material 3 feet thick. A portion of the alignment was in hard shale and sandstone that made trimming operations difficult. In rocky areas, the canal was overexcavated 3 inches and backfilled with pervious material. Pervious backfill was used so as not to seal off any ground water flow in the formation and thus build up hydrostatic pressures against the canal lining. Where expansive clay was encountered, it was overexcavated 3 feet and backfilled with impervious soil.

The major portion of concrete lining was placed with a paving machine especially manufactured for use on this project (Figure 186). The liner began operations on February 20, 1967 at Milepost 0.2 and completed operations on May 20, 1967 at Milepost 14.8. The liner followed closely behind a trimmer, and both machines used the same set of guide wires. Finishing of the lining was done from a jumbo that followed closely behind the liner. A membrane curing compound was sprayed over the surface of the lining from a second jumbo.

Longitudinal joint sealant was of the polyvinyl chloride type and was placed by the lining machine. The transverse joints were filled with a two-component, polymer-type, plastic, joint sealant locally mixed. The procedure was for a crew to chip out the area where the transverse joints crossed the longitudinal joints and any other irregularities, sandblast the transverse joint to remove the curing compound, and then apply the sealant. The sealant was transported on a small trailer where it was mixed and heated. Then, it was pumped to two hand-held application guns and applied by operators who walked up and down the slopes of the canal. Sealant operations began on August 21, 1967 and were completed on November 15, 1967.

Concrete control was maintained by the proper proportioning, mixing, placing, finishing, and curing of all concrete incorporated in the work. Slump tests were taken at the batch plants and also at the placement site. The placement itself was under constant surveillance by field inspectors.

Batching and mixing were done at three plants: the first was located near the Coastal Branch turnout, the second near Badger Hill Pumping Plant, and the third

TABLE 15. Major Contracts—Coastal Branch

	Specification	Low bid amount	Final contract cost	Total cost—change orders	Starting date	Completion date	Prime contractor
Preconsolidation Mile 10.8 to Mile 14.8	65-55	$81,288	$80,152	$2,277	12/22/65	2/25/66	Eugene Luhr & Co.
Aqueduct—Avenal Gap to Devil's Den Pumping Plant Mile 0 to Mile 14.8	66-04	3,674,659	4,215,711	227,671	3/24/66	5/29/68	Fredrickson & Watson Construction Co.

near Las Perillas Pumping Plant, later moved to the vicinity of Devil's Den.

Initial Operations

Operation of the Coastal Branch, California Aqueduct, began in January 1968. Settlement of the Badger Hill Pumping Plant headworks occurred during the first year, resulting in replacement of three badly damaged concrete panels.

A heavy local rain during November 1972 caused an overchute to plug with debris. Subsequent floodflows topped the concrete lining from the outside and destroyed two panels.

In October 1973, the lining in Pool 1 was raised approximately 2 feet to allow fluctuations of water levels in the California Aqueduct during on-peak/off-peak operations.

Figure 186. Paving Train

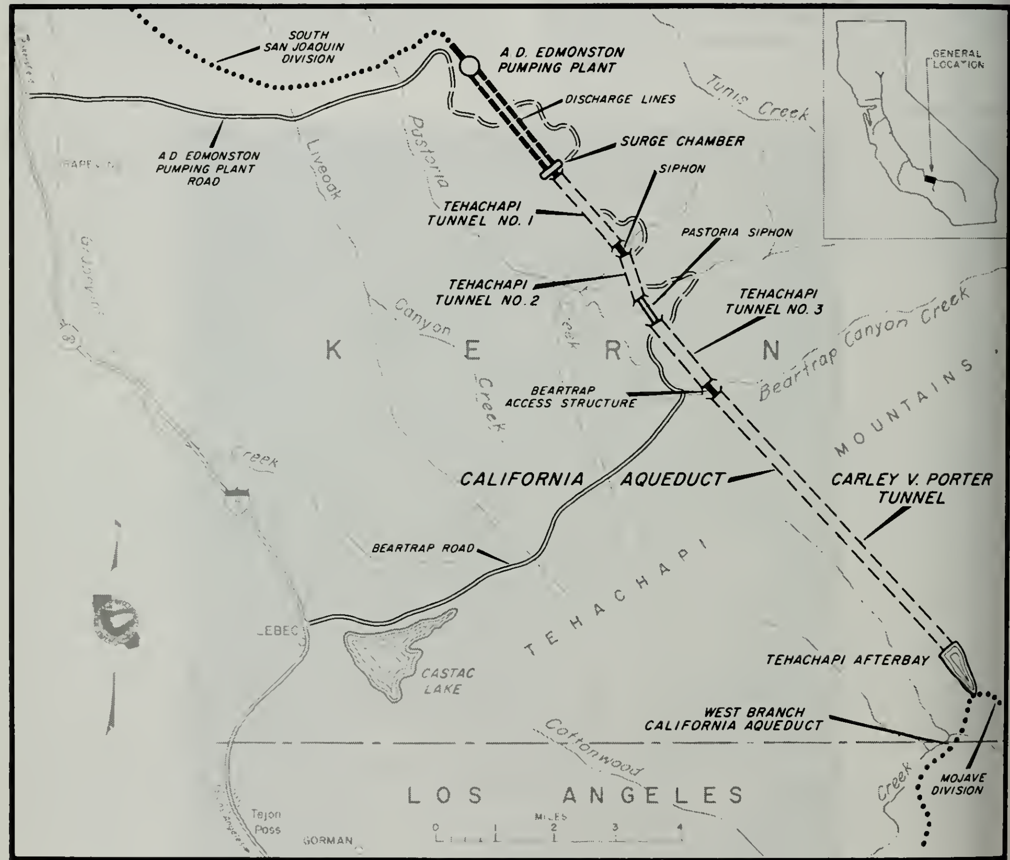

Figure 187. Location Map—Tehachapi Division

CHAPTER VIII. TEHACHAPI DIVISION

Introduction

Role in the State Water Project

The Tehachapi Division is one of the most noteworthy components of the State Water Project. The engineering solution to raising 4,410 cubic feet per second of water through a single lift of 1,926 feet and tunneling 8 miles of the Tehachapi Mountains under adverse geologic conditions represents a major accomplishment. The crossing of this great barrier provides the vital link that conveys water from Northern California to an arid Southern California.

The Tehachapi Division extends from A. D. Edmonston Pumping Plant southeasterly through Tehachapi Afterbay. The Afterbay marks the end of the Tehachapi Division and the beginning of the Mojave Division and the West Branch (Figure 187).

Geography, Topography, and Climate

The Tehachapi Mountains are the link between the Transverse Ranges near the ocean and the Sierra Nevadas to the east and form an arcing mountainous barrier between the San Joaquin Valley and Antelope Valley. Individual peaks in the Tehachapi Mountains are not particularly high, with only a few over 5,000 feet in elevation. However, the terrain is rugged, and there is little access into the Mountains except by fire trails or an occasional private road serving the local cattle ranches.

This is a semiarid region with rainfall averaging less than 10 inches annually. However, intense storms with high precipitation or snowfall amounts are not uncommon.

There are no major streams in the area. Most drainage courses on both sides of the crossing are located in steep-walled canyons and are ephemeral streams. The only two watercourses of concern to the conveyance facilities are Pastoria Creek and Cottonwood Creek. Branches of Pastoria Creek are crossed in two separate canyons, one at Pastoria Siphon and the other at the Beartrap Access Structure. Pastoria Creek also crosses the California Aqueduct and main access road just upstream of A. D. Edmonston Pumping Plant in the northern foothills of the Tehachapis. Cottonwood Creek is the local expression of some of the drainage from the Tehachapis to the south into Antelope Valley. The south portal of the Carley V. Porter Tunnel and Tehachapi Afterbay are located in the flatter slopes of Cottonwood Canyon, which contains Cottonwood Creek.

Temperatures in the Tehachapis reach 100 degrees Fahrenheit in the summer. During the winter, especially in the higher elevations, freezing weather is common in conjunction with snowfalls.

Features

The 10.6-mile length of the Tehachapi Division encompasses A. D. Edmonston Pumping Plant (formerly called Tehachapi Pumping Plant), two underground discharge tunnels, four tunnels with pipe conduits between portals, and Tehachapi Afterbay on the southern side of the Mountains. The Pumping Plant consists of two bays of pumps with two separate underground discharge lines joining at the surge tank, 1,926 feet above the pumps. From here, the flow then crosses the Tehachapi Mountains by gravity (Figure 188).

A. D. Edmonston Pumping Plant, discharge lines, surge tank, and the connection from the surge tank to Tunnel No. 1 are discussed in Volume IV of this bulletin. The remaining Tehachapi facilities are covered in this chapter. Also included in this chapter, for continuity of construction contracts, are those portions of the Mojave Division and the West Branch that lie within the limits of construction of Tehachapi Afterbay as shown on Figure 187. Statistical summaries of Tehachapi Division conveyance facilities to the bifurcation and certain West Branch and Mojave Division conveyance facilities beyond the bifurcation are presented in Tables 16 and 17.

Tunnel No. 1, just downstream of the surge tank, is 7,933 feet in length and is connected to Tunnel No. 2 by 242 feet of cast-in-place concrete pipe. Tunnel No. 2 is 2,810 feet long and is connected across a canyon to Tunnel No. 3 by the 2,452-foot-long, Pastoria Creek, steel-pipe siphon. Tunnel No. 3 is 5,709 feet in length and is connected to Tunnel No. 4 by 315 feet of cast-in-place concrete pipe designated the Beartrap Access Structure.

Tunnel No. 4 was renamed the Carley V. Porter Tunnel in honor of the coauthor of the Burns-Porter Act and is both the longest, 25,075 feet, and the final tunnel in the Tehachapi crossing. Flow discharges from the south portal of Carley V. Porter Tunnel through a gated control structure to Tehachapi Afterbay.

The Afterbay is a trapezoidal canal section slightly over 3,300 feet long. Its outlet is both the geographical end of the Tehachapi Division and the takeoff point for the West Branch of the California Aqueduct.

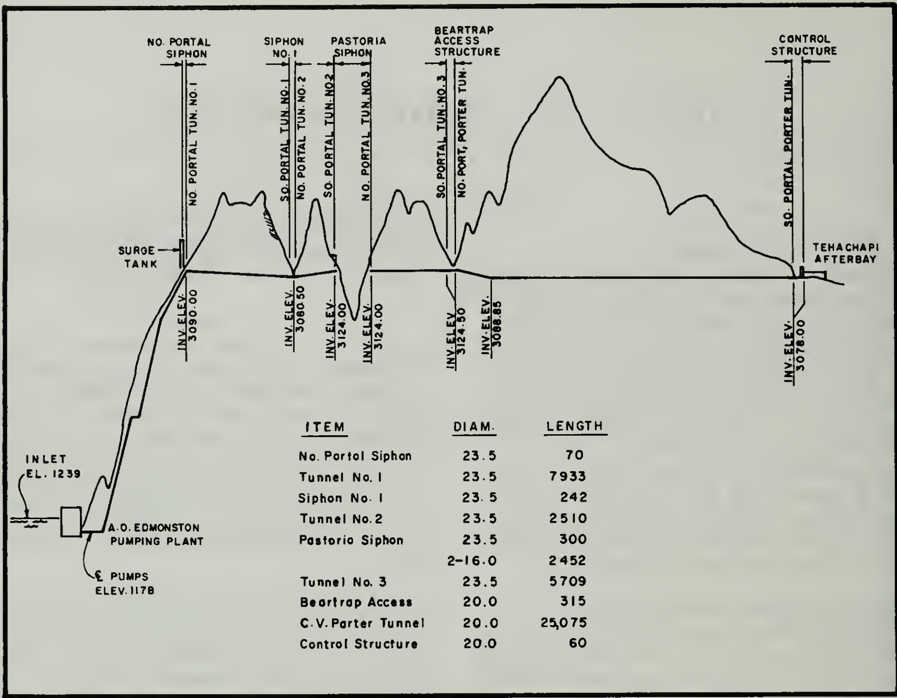

Figure 188. Schematic Profile—Tehachapi Crossing

TABLE 16. Statistical Summary of Tehachapi Division to Bifurcation

Aqueduct Reach	Type of Conveyance	Inside Diameter (Feet)	Capacity (Cubic feet per second)	Length (Miles)
Tunnel No. 1	Lined tunnel	23.5	5,360	1.5
Siphon No. 1	Reinforced-concrete conduit	23.5	5,360	0.1
Tunnel No. 2	Lined tunnel	23.5	5,360	0.5
Pastoria Siphon				
First Barrel	Existing steel conduit	16.0	2,680	0.5
Second Barrel	Planned conduit	--	2,680	0.5
Tunnel No. 3	Lined tunnel	23.5	5,360	1.1
Beartrap Access Structure	Reinforced-concrete conduit	23.5 to 20.0	5,360	0.1
Carley V. Porter Tunnel	Lined tunnel	20.0	5,360	4.7
Afterbay	Lined channel	*	5,360	0.3
Canal	Lined channel to bifurcation	†	5,360	0.3

AQUEDUCT FEATURE

Check Structure: 1 two-radial-gate structure at end of afterbay
Wasteways: 2, one at Siphon No. 1 and one at low point of Pastoria Siphon
Blowoffs: 2, one at Siphon No. 1 and one at Beartrap Siphon

OPERATIONS

Flow control from A. D. Edmonston Pumping Plant control center with area control from San Joaquin Field Division; wasteway control manual on-site operation; afterbay gates (when installed) local automatic and manual, with area control from Castaic Operations and Maintenance Center

* Channel Data: Bottom width, 50 feet; water depth, 22 feet; side slopes, 3:1
† Channel Data: Bottom width, varies from 50 to 32 feet; water depth, 22 feet; side slopes vary from 3:1 to 2:1

TABLE 17. Statistical Summary Beyond Tehachapi Division Bifurcation

WEST BRANCH CANAL

Type
Concrete-lined—trapezoidal

Dimensions
Lined depth, 19.5 feet; bottom width, 24 feet; side slopes, 2:1; length, 1.5 miles from bifurcation to Oso Pumping Plant

Capacity
3,129 cubic feet per second

Freeboard
3.0 feet lined and a minimum of 2.8 feet of earth berm above lining

Lining
4-inch unreinforced concrete—sealed longitudinal and transverse contraction joints on a maximum of 12½-foot centers

SIPHON

Box culvert siphon at Tejon Ranch access road

MOJAVE DIVISION CANAL

Type
Concrete-lined—trapezoidal—checked

Dimensions
Lined depth, 24.5 feet; bottom width, 10 feet; side slopes 2:1; length, 1 mile from bifurcation to Cottonwood Chutes

Capacity
2,388 cubic feet per second

Freeboard
2.5 feet lined and 3.0 feet of earth berm above lining

Lining
4-inch unreinforced concrete—longitudinal and transverse contraction joints on a maximum of 12½-foot centers

Check Structure
1 two-radial-gate structure

Cross-Drainage Structure
1 overchute

SIPHON

One at Chute No. 2

The West Branch beyond the Afterbay is a trapezoidal canal section and extends southwest 7,500 feet to the Oso Pumping Plant forebay. The continuing California Aqueduct is a similar canal section. It extends southward for 2,100 feet before dropping 130 feet in elevation through two energy-dissipating chutes and stilling basins.

Tunnels Nos. 1, 2, and 3 (Figure 189) were driven as horseshoe sections and then lined to a final circular section. The inside diameters of Tunnels Nos. 1, 2, and 3 and the connection between Tunnels Nos. 1 and 2 are 23.5 feet. The Carley V. Porter Tunnel (Figure 190) is 20 feet in diameter and was driven as a circular section. The Beartrap connection forms a transition from 23.5 to 20 feet. The Pastoria Siphon presently is a single barrel 16 feet in inside diameter. A second barrel will be added in the future.

Flow regulation in the Tehachapi Division is minimal because the system is designed for gravity flow downstream of the surge tank and is governed only by the natural energy gradeline of the system. The free water surface in the surge tank rises only to the elevation required to pass the pump delivery through to Tehachapi Afterbay. At low flow, minor control is exercised to fill the tunnels for hydraulic efficiency by using twin radial gates at the beginning of the Afterbay, as described later in this chapter in the section on hydraulic criteria.

Geology and Soils

The Tehachapi Mountains extend southwest from the Sierra Nevada, forming most of the mountain barrier around the south end of the San Joaquin Valley. The central portion of the Mountains is comprised of older crystalline rocks consisting mainly of a diorite complex, quartz monzonite, schist, granite, and a few older metamorphic rocks. These crystalline rock types are, for the most part, in fault contact with one another. Although the higher peaks in the vicinity of the aqueduct crossing are only about 5,000 feet in elevation, the Mountains are rugged and pose a formidable topographic barrier. Younger sedimentary and volcanic rocks lap onto both the north and south flanks of the mountain ranges. On the south flanks, upper Miocene sandstones and shales of the Santa Margarita formation and Pleistocene lakebeds, consisting of siltstones and claystones, lap onto the southern flanks of the mountain ranges. On the north side of the Mountains, sandstones of the Eocene Tejon formation are in depositional contact with the diorite complex. These in turn are overlain by the Oligocene-Miocene sedimentary and volcanic rocks of the Tecuya formation. During the complex tectonic history of the mountain range, periods of uplift have tilted and deformed the younger rocks on both the north and south flanks.

Geologic structures in the Tehachapi Mountains are dominated by the Garlock fault, the second largest fault in California. In the vicinity of the aqueduct crossing, this fault splits into two branches, the North and South Garlock faults. In between the branches of the Garlock fault is a fault block of pre-Cambrian series of schists and quartzites called the Pelona schist. The Garlock fault separates two different rock types in the vicinity of the aqueduct crossing: north is a diorite complex and south is the Tejon Lookout granite with some roof pendants of Paleozoic marbles and hornfels. A short distance west of the tunnel crossing is a thrust fault, the Pastoria thrust, whose thrust plate comprises badly broken and decomposed quartz monzonite.

The northwesterly trending San Andreas fault crosses the west end of the Tehachapi Mountains and passes to the south, about 5½ miles from the south portal of the tunnel system. Numerous other smaller

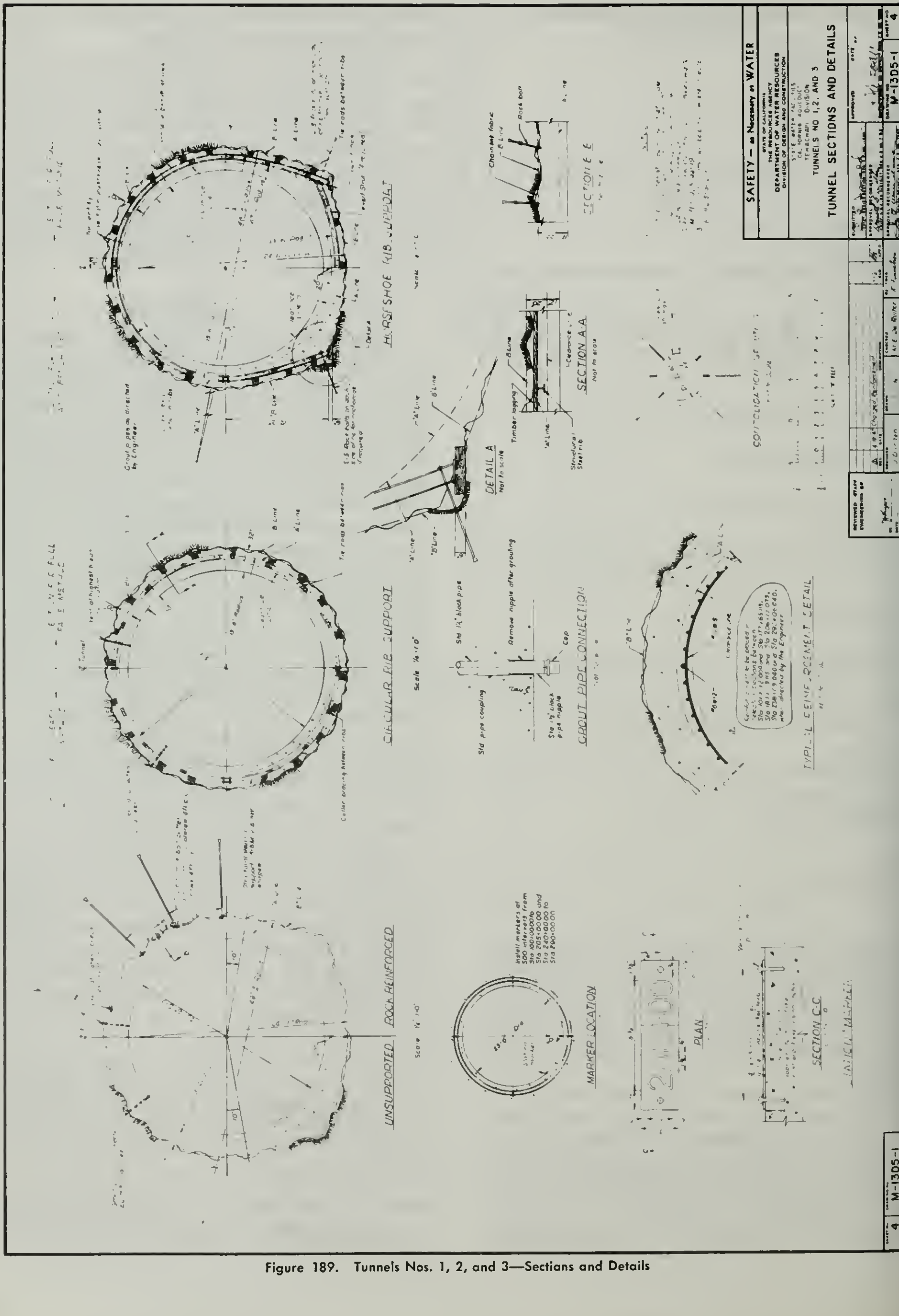

Figure 189. Tunnels Nos. 1, 2, and 3—Sections and Details

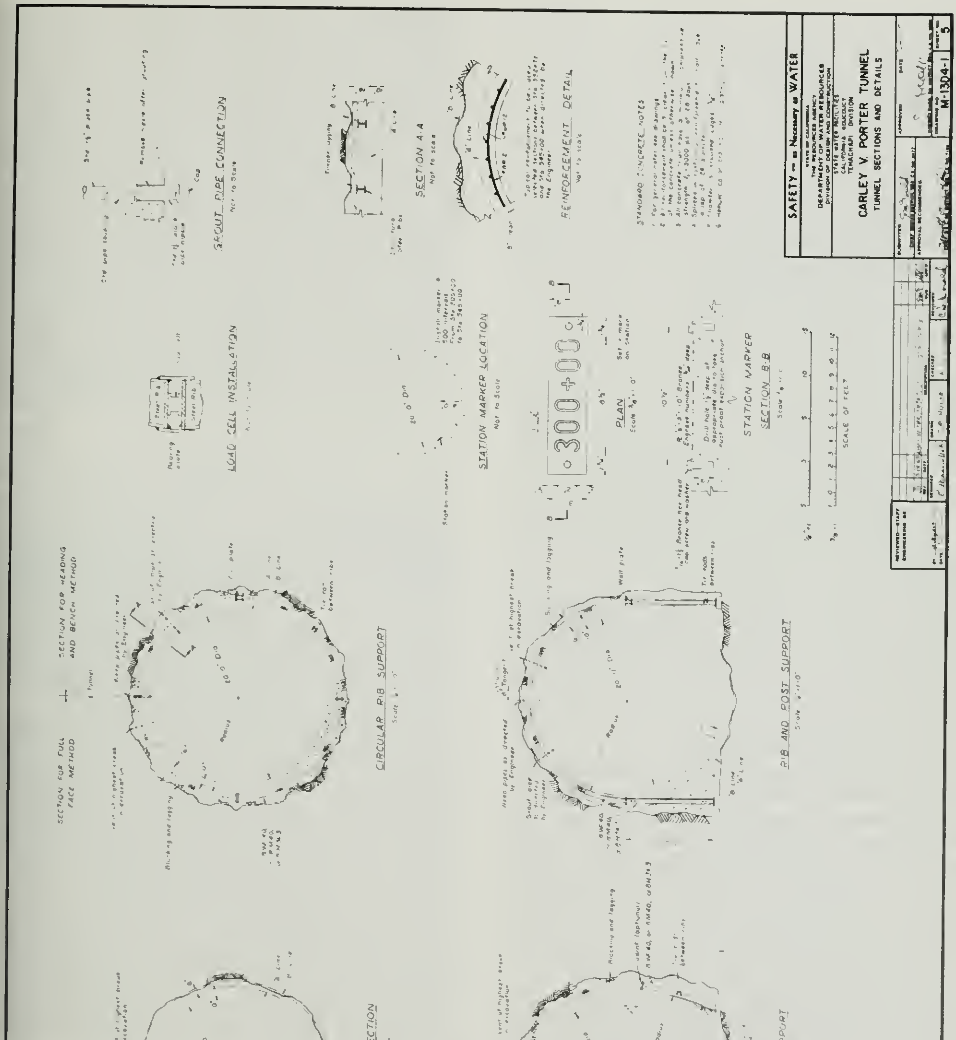

Figure 190. Carley V. Porter Tunnel—Sections and Details

unnamed faults are present throughout the mountain block.

Early studies of Tehachapi Mountain crossings were made of two main groups of alternatives: (1) high-level crossings which would require a relatively high pump lift at the south end of the San Joaquin Valley but would have relatively short tunnels through the mountains, and (2) low-level crossings which would require a relatively low pump lift at the south end of the San Joaquin Valley but would have required very long expensive tunnels. In all, five high-level alternatives were studied with elevations at the north portal ranging from 3,000 feet to 3,600 feet and two low-level tunnel alignments with north portal elevations of 1,500 and 1,870 feet.

The long tunnel alignments were up to 26.7 miles long and penetrated the Garlock, San Andreas, Liebre, and Clearwater fault systems at considerable depth. The great depths of cover, adverse tunneling conditions, large construction cost and, most important of all, the possibility of movement on one of the major fault systems causing damage extremely difficult to repair detracted considerably from the low-level alignments. However, the cost of lifting water at the north slope of the Tehachapi Mountains forced consideration of the possibility of using a low-level alignment.

In 1953, geologic exploration started on a high-level crossing just west of Pastoria Creek. Parts of this crossing were in the vicinity of the Pastoria thrust fault, and the quartz monzonite in the thrust plate was so badly broken and decomposed that it would make tunneling extremely difficult. For this reason, alignments were shifted to the east side of Pastoria Creek to avoid the bad rock conditions and, in 1956, geologic investigations started east of Pastoria Creek. Three alignments were investigated in this area and one higher alignment at elevation 3,600 feet farther west near Gorman. A policy decision was made to cross major active faults at or near ground surface so rapid repairs could be made if the faults moved. This decision precluded further consideration of low-level alignments. An alignment at 3,090 feet elevation was finally selected as one that struck the best balance between tunneling conditions and at the same time crossed the Garlock fault zone at or near the surface.

The finally selected alignment consisted of four tunnels, named from north to south, Tehachapi Tunnels Nos. 1, 2, and 3 and the Carley V. Porter Tunnel. Tunnels Nos. 1 and 2 penetrate the diorite complex on the north slope of the Tehachapi Mountains. These two tunnels encountered the most favorable tunneling conditions in the mountain block and penetrated fractured relatively competent diorite, diorite gneiss, and occasional schist. Minor faulting was present but ground water flows were not a problem.

Tunnel No. 3 penetrated the fault block of Pelona schist between the north and south branches of the Garlock fault. Movement of the fault block between the two large faults generated many smaller faults within the block. The resulting faulted and crushed rock in the fault zone made tunneling conditions substantially worse than in Tunnels Nos. 1 and 2.

In the southernmost and longest tunnel in the Tehachapi crossing, the Carley V. Porter Tunnel, the worst tunneling conditions were encountered. The Tejon Lookout granite south of the Garlock fault was severely fractured by movements of both the Garlock and San Andreas faults. Water percolating into these fracture systems had caused the granite to decompose badly in many places, with the result that the hardness of the rock varied greatly from place to place, ranging from soft running ground to occasional hard ribs of granite. Extremely high-pressure water, at times, was encountered at the tunnel face. The north portal of this tunnel was in the main branch of the Garlock fault, and the southern portion penetrated about 1,500 feet of Pleistocene lakebed deposits consisting primarily of silts and clays.

Design

Alignment Criteria

As discussed in the foregoing sections, the Department of Water Resources expended great effort in planning the aqueduct crossing of the Tehachapi Mountains. Not only was it one of the most important features, but it presented major problems because of the difficult geological and hydraulic conditions. Solutions to these problems required formulation of basic criteria for the horizontal and vertical alignments of the conduit and the hydraulic conditions.

Design studies showed that the Tehachapi crossing required long tunnel reaches to convey water with reasonable economy. The shortest route lay across a major seismic zone, the Garlock fault. The alignment was also close to the San Andreas fault. This situation was not unique since two other principal water conveyance systems for Southern California, the Los Angeles Aqueduct and the Colorado River Aqueduct, cross the San Andreas fault by tunnels.

To minimize exposure of yet another water supply tunnel to earthquake damage and to maximize safety and reliability, it was decided that all major faults would be crossed either at the ground surface or at very shallow depths. This would afford the greatest ease of repair, in case of damage resulting from movement along a fault.

This decision for rapid access fixed the alignment of the south portion of the Tehachapi crossing. To meet the approved criteria, the northern portal of the Carley V. Porter Tunnel was set in Beartrap Canyon, very close to the surface exposure of the Garlock fault. The south portal was set to accommodate a canal at the elevation of Antelope Valley.

The Metropolitan Water District of Southern California requested that other alternatives be examined for the location and design of Tehachapi Pumping

Plant. They suggested that a two- or three-stage pump lift might be preferable to the single-lift pumping plant under study by the Department. As a result, additional studies were made to evalute the feasibility of multilift pumping schemes. Results of these studies confirmed the Department's earlier conclusions that a single lift was the optimum arrangement for pumping water over the Tehachapi Mountains. The Department's Bulletin No. 164 is a compilation of the reports and resumés used to support this decision. This decision fixed the alignment of the Tehachapi crossing.

Several criteria were satisfied by the selected tunnel alignment. The route from the Pumping Plant to Beartrap Canyon was nearly straight and thus a minimal distance. Two additional headings were available between Tunnels Nos. 1 and 2 for faster excavation. North Garlock fault was crossed at ground level with Pastoria Siphon.

Tunnels

The general design of project tunnels is discussed in Chapter I of this volume. Rock quality in the Tehachapi tunnels was expected to deteriorate from north to south, and substantial support was expected to be necessary. To permit flexibility, the driving configuration and method were made optional to the contractor. Pay lines for excavation and concrete were designated for various alternatives.

A permanent portal section was not required where cast-in-place pipe connected the tunnels. Specifications required temporary portal support only during the tunnel-driving period. An exception to this requirement was at the north portal of Tunnel No. 3, where its location at the Garlock and Pastoria faults and the timing of the Pastoria Siphon construction required a permanent portal section. For integrity at the connections between the tunnels and pipe sections, the tunnel lining was reinforced both horizontally and circumferentially for a distance from the portals of seven times the radius.

As a precautionary measure against possible ground water intrusion into the lined tunnels, waterstops were placed in the lining joints 150 feet from each portal where reinforcement steel was not extended through the waterstops. Tunnel driving was required from all portals simultaneously. Separate portal excavation contracts or adjacent facility contracts were utilized where feasible.

Hydraulic Criteria

The hydraulics of the crossing system are influenced by A. D. Edmonston Pumping Plant, Tehachapi Afterbay, and hydraulic phenomena in the conduits. Several design and operational criteria were established within those constraints.

1. Rates of flow in the crossing system will vary in increments of 315 cubic feet per second (cfs) (the capacity of each A. D. Edmonston pump) up to 4,095 cfs (combined capacity of 13 pumps—one unit is expected to be on standby for routine maintenance). Only 11 of the ultimately planned 14 units were installed initially at A. D. Edmonston Pumping Plant.

2. Operating water surface elevations of Tehachapi Afterbay were established between elevations 3,095 and 3,100 feet.

3. The Manning roughness coefficient, "n", for tunnels and cast-in-place siphons was assumed to range from 0.014 to 0.011 according to what assumption would promote the most critical condition for the hydraulic phenomena under investigation. The "n" for Pastoria Siphon was assumed constant at 0.012.

4. The invert profile of Carley V. Porter Tunnel was designed with a short initial reach on a super critical slope followed by a long flat slope to the Afterbay. A straight profile was not used because unfavorable hydraulic conditions would result from entrapment of air since both Tunnel No. 3 and Carley V. Porter Tunnel were partially full when less than 11 pumps were operating at A. D. Edmonston Pumping Plant. Two radial gates were installed at the Afterbay to force a hydraulic jump to occur on the steep slope, close to the inlet portal vent structure, and allow gravity flows between 315 cfs and 5,400 cfs.

5. Spill was prohibited at any structure, including the A. D. Edmonston Pumping Plant surge chamber, the two vent structures at Pastoria Siphon, and the air vent at the upstream grade break of the Carley V. Porter Tunnel.

6. At Beartrap Canyon, a 30-inch outlet was installed to accommodate a future turnout for local water users.

Construction

Construction supervision was administered from the project office, initially located in the San Fernando Valley and subsequently relocated to Castaic. Local trailer offices for field engineers and inspectors were set up close to the contractor's headquarters which were near the tunnel portals.

General information about the major contracts for the construction of the facilities for the Tehachapi Division is shown in Table 18.

Construction is described in this chapter on a north-to-south basis. A major exploratory tunnel contract for Carley V. Porter Tunnel is included in the description for that tunnel. Construction of the roads and minor exploration work are discussed separately.

Since tunnel driving is more hazardous than most construction work, safety was stressed and achieved by the constant effort of the Department's forces. Specifications required a first-aid station with an ambulance in attendance onsite. Division of Industrial Safety personnel also enforced safe work habits and helped minimize hazardous working conditions.

A strict fire prevention program and construction site personnel restriction were specified in the construction contracts.

TABLE 18. Major Contracts—Tehachapi Division

	Specification	Low bid amount	Final contract cost	Total cost—change orders	Starting date	Completion date	Prime contractor
Tehachapi Access Roads	63-19	$559,598	$570,880	$9,527	7/ 9/63	6/17/64	William H. Schallock,
Tunnels Nos. 1, 2, and 3 and the Pastoria Access Road Mile 294.9 to Mile 298.6	66-37	29,394,648	30,433,674	1,285,744	10/31/66	12/15/69	Granite Construction Company, Gates and Fox Company, Inc., and Gordon H. Ball Enterprises
Pastoria Siphon Mile 297.0 to Mile 297.5	68-29	3,448,523	3,618,138	16,550	1/22/69	3/ 5/71	Gates and Fox Company, Inc., Granite Construction Company, and Gordon H. Ball, Inc.
Beartrap Access Structure Mile 298.6 to Mile 298.7	70-27	451,66 0	459,471	16,128	9/23/70	5/18/71	Dravo-Atkinson-Groves
Carley V. Porter Tunnel Phase I—Mile 298.7 to Mile 299.3	64-34	1,170,604	1,353,382	64,720	7/22/64	6/ 8/65	J. F. Shea Co., Inc.
Carley V. Porter Tunnel South Portal Excavation	65-04	677,839	659,715	--	3/29/65	11/15/65	A. A. Baxter Corporation
Carley V. Porter Tunnel Main Contract Mile 298.7 to Mile 303.4	65-29	33,788,800	48,316,215	4,579,214	9/ 7/65	11/20/70	Dravo Corporation, Guy F. Atkinson Company, and S. J. Groves & Sons
Tehachapi Afterbay Mile 303.4 to Mile 304.0	69-29	5,163,261	5,587,317	52,085	1/24/70	11/15/71	Zurn Engineers
Site Development Cottonwood Powerplant	66-46	1,204,216	1,224,446	8,000	11/17/66	7/18/67	Altfillisch-Fulton Co.
Site Development Oso Pumping Plant	67-07	447,843	472,713	13,277	3/10/67	11/15/67	W. E. McKnight Construction Company

Subsurface exploration in the Tehachapi Mountains commenced during the early planning of the State Water Project. Beginning in 1957, a pioneer road was constructed near the probable alignment of the Carley V. Porter Tunnel. Limited access to other possible routes was available by ranch roads of the Tejon Ranch Company.

In the early 1960s, as part of the subsurface exploration program, two short adits were driven above the final portal elevations of Tunnel No. 3 and the Carley V. Porter Tunnel. These adits were maintained for inspection by bidders on the tunnel contracts.

Design and Construction by Contract

Tehachapi Access Roads

In June 1963, a contract was awarded to the William H. Schallock Company for two surfaced access roads. The Grapevine access road commenced at the foot of the "Grapevine Grade" of U. S. Highway 99 (now Interstate 5) and extended along the northern Tehachapi foothills to the site of A. D. Edmonston Pumping Plant. Beartrap access road originated at Lebec on Highway 99 and extended eastward to the head of Beartrap Canyon. This road provided access to the south portal of Tunnel No. 3 and the north portal of the Carley V. Porter Tunnel.

A pioneer road, the Pastoria Road, also was included in the Schallock contract and extended from the A. D. Edmonston Pumping Plant site to the vicinity of the north portal of Tunnel No. 1. The contractor for the excavation of A. D. Edmonston Pumping Plant improved the drainage of this pioneer road and connected it with the Grapevine access road, thereby providing a loop road net for the construction of the Tehachapi crossing facilities.

Tunnels Nos. 1, 2, and 3 and Pastoria Access Road

Design. The tunnels and the cast-in-place siphon were sized assuming a Manning's "n" of 0.014, and all are of the same finished inside diameter. The invert of Tunnel No. 3 was set to clear the North Garlock fault at the surface and sloped to drain the tunnel through Pastoria Siphon.

The design head on Tunnels Nos. 1 and 2 was 90 feet and on Tunnel No. 3 was 46 feet. Siphon No. 1 and the cast-in-place pipe section from the north portal of Tunnel No. 1 to the surge tank were designed for a head of 120 feet and an external earth load of 10 feet.

The permanent north portal structure for Tunnel No. 3 was designed as a counterforted retaining wall. The embankment for Pastoria Siphon was designed for a soil density of 120 pounds per cubic foot having a cohesive strength above the phreatic line of 1,200 pounds per square foot (psf) and 400 psf below the phreatic line. A seismic acceleration factor of 0.2g was used and the stability of the embankment analyzed by the Swedish Slip Circle method of analysis. The spillway and channel draining Pastoria Creek over the embankment were designed for a 500-year storm having a maximum discharge of 3,000 cfs. Riprap protects the toe of the fill.

Siphon No. 1 contains provisions for draining Tunnels Nos. 1 and 2 by means of a 24-inch steel pipeline located in the invert at the low point of the siphon. Flow is controlled by twin, 24-inch, butterfly valves located in a valve vault connected to the siphon barrel. The access manhole to the vault is made from 48-inch-diameter, precast, concrete, pipe sections extending through the embankment. The drain line discharges downslope about 360 feet in an energy-dissipator box to a stone-protected channel.

Construction. The north portal of Tunnel No. 1 was excavated as part of the A. D. Edmonston Pumping Plant intake channel contract (discussed in Chapter VI of this volume). The exit portal for the underground discharge lines was excavated at the same time.

The north portal cut was 150 feet deep with the rock excavated in a series of $1\frac{1}{2}$:1 slopes with four intermediate 15-foot benches. Tractors and rippers were used. Benches were shaped to drain in both directions. The exposed diorite gneiss was moderately weathered to decomposed.

Two shear zones were exposed by the portal cut. The thickness of the zones varied from hairline cracks to sections of contorted, sheared, and altered material nearly 20 feet thick. The portal cut contractor was not required to provide portal protection for the vertical rock face for the initial tunnel excavation. The tunnel contractor found the vertical cut unstable on the face of the tunnel heading due to the intersection of the foliation and joint shears which dipped out of the slope.

Rock bolts, steel mesh, and steel channel walers were installed from just above the crown of the tunnel to the first bench in the portal cut. Shotcrete was applied up to the level of the second bench. The drainage ditches for the cut-slope berms were lined with concrete, and the drainage away from the portal face was improved. However, blasting in the tunnel produced cracks up to $2\frac{1}{2}$ inches wide in the slope above the tunnel portal cut. An 0.8-inch coating of asphalt was placed over the cracks.

Because of these unstable portal conditions, the portal collar section was moved toward the surge tank, the specially reinforced area was extended 48 feet farther into the tunnel, and a second net of reinforcing bars was added.

The contractor for Tunnels Nos. 1, 2, and 3 improved Pastoria Road sufficiently to permit hauling of heavy materials and the passage of high-volume vehicle traffic. Grades were as much as 12% with 100-foot-radius curves. Maintenance and relocation of Pastoria road, as required for construction in the Tehachapi Division, were sequentially assigned to various contractors during the construction period.

The south portal of Tunnel No. 1 was excavated at the same time as the north portal of Tunnel No. 2. The horizontal distance between the two portals is less than 200 feet; therefore, stripping for Siphon No. 1 also was included. A total of 239,800 cubic yards was excavated for the siphon and the two portals.

The portal of Tunnel No. 1 with a cut slope of $\frac{3}{4}$:1 required only one 15-foot bench. The north portal of Tunnel No. 2 was much steeper, requiring six benches altogether. One bench was widened to 30 feet for realignment of Pastoria Road. Two strong and one weak joint patterns and a strong reverse fault were exposed on the face of the excavation.

Before facing off the tunnels, the portals were protected with chain-link fabric and rock bolts.

A small (30 cubic yards) slide occurred just to the right of the south portal to Tunnel No. 1. The slide area was cut back without further difficulty.

Tunnel No. 1 (7,993 feet long) was first driven from the south portal. Excavation from the north portal started approximately seven months later. The tunnel was driven in a horseshoe-shaped section. The rock was diorite gneiss throughout and varied from hard foliated, which would stand unsupported, to completely crushed, which required preconsolidation grouting, breastboarding, and pilot drifts. Little ground water was encountered.

In general, the driving of Tunnel No. 1 was not difficult. Average blasting required 125 holes varying from 3 to 10 feet in depth. The heading, with a minor exception at the north end, was carried full face. The heading from the south was driven 4,353 feet in 132 working days. The north heading required 132 days to hole through after driving for 3,580 feet. A typical round amounted to 250 pounds of Gelex No. 2 powder. Three shifts a day for five days a week were employed.

The conventional drilling jumbo, work jumbo for placing the steel bracing, mucking machine, and muck train were used for excavation. The tunnel bracing consisted of horseshoe-shaped sets of 8-inch, wide-flanged, four-piece, horseshoe-shaped ribs. The sets were supported longitudinally with collar braces. A typical set had 12 steel collar braces. Timber blocking and lagging secured the sets and provided further support between the rock and the support system in weak areas.

The contractor requested permission to use shotcrete in place of or as additional direct support where blocking had already been placed. Reasons cited for the request were: (1) smooth wall drilling techniques could be used, (2) greater safety for personnel, (3) less costly than steel or wood lagging, and (4) reduction of contact grouting behind tunnel lining. After a test section was tried and compression tests made on the shotcrete, the request was approved.

Rock bolts 1 inch in diameter and 10 feet in length, with an expansion steel anchor and a $\frac{3}{8}$-inch-thick steel bearing plate, occasionally were installed. Two-inch steel mesh also was used as a supplement. Where necessary, additional support was provided by 30- to 60-pound steel rail, 4- and 8-inch steel channels, No. 11 reinforcing steel, and heavier timber blocking and lagging.

The tunnel driving from the north portal encountered poorer rock, especially in the first few hundred feet. A fault zone about 35 feet wide was encountered 200 feet in from the portal and breastboarding and benching were required. About 2,000 feet farther in a fault zone supported only by shotcrete, material slipped out resulting in a 300-cubic-yard fallout. Steel sets were placed on 4- and 6-foot centers about 60 feet back from the fault. A timber bulkhead was placed just behind the caved area, and almost 700 cubic yards of concrete was pumped ahead of the bulkhead. The contractor then mined through the resulting concrete plug and continued normal tunnel driving. The caved area later was consolidated and contact-grouted.

Ground water was never a problem, being limited to that contained in joints and fractures. Generally, the fractures and joints were tight or filled with alteration products, such as clay or chlorite. Locally, in areas where the fractures and joints were open, many seeps and drips occurred. Where flows occurred, they dried up rapidly, usually within two weeks. The maximum flow was 140 gallons per minute (gpm) from the release of locally stored water. Except near the portals, precipitation did not have any appreciable effect on the quantity of seepage. The maximum cover near the mid-length of the tunnel was 660 feet.

An instrumentation program for determining rock stresses and strains was established in July 1967 and terminated in March 1968. Three types of instruments were used: load cells, extensometers, and a seismitron. These instruments allowed estimates of the actual loads in the rock and on the tunnel support system in order to safely construct the tunnel.

The load cells measured the compressive force exerted on a pillowlike cell which was placed under the leg of the set during the regular bracing cycle. Six cells were installed but half were damaged by mucking or blasting. Readings were made with a wheatstone bridge. The maximum load recorded was 84,000 pounds, or about 25% of the rib capacity.

The extensometers measured the amount of strain (relaxation) in the rock around the tunnel bore. Both single- and multiple-point readout extensometers were used. The single-point instrument, 15 feet in length and similar to a rock bolt, measured the difference in strain between the collar and the bottom of the hole which was assumed to be outside the zone of relaxation of the rock. The measured values ranged from 0.065 to 0.420 of an inch where the steel sets were on 5- or 6-foot centers. The strains generally stabilized by the time the heading had advanced 150 to 200 feet beyond the instrumented stations.

The multiple extensometers recorded strains at lengths of 2, 3, 4, 5, 10, 20, 30, and 50 feet from the collar of the hole. These holes were located in the roof of the tunnel in shotcreted sections. Readings of only 0.02 of an inch relaxation at 2-foot depths and 0.05 of an inch at 5 feet indicated that the shotcrete had effectively supported the rock. An extensometer in the area of the fallout previously described was destroyed before any readings were taken. However, extensometer readings 50 feet back from the fallout indicated the roof deflected downward and caved in only when the shotcrete failed to provide sufficient restraint.

Prior to holing through from the north heading, excavation from the south heading was discontinued, and concreting of the invert was started at the south portal. A conventional batch plant was set up at the portal. Aggregate and cement were trucked in. The invert lining was placed in 21-foot sections and, on holing through the tunnel, concreting of the invert was extended to the north portal. The concrete was hauled into the tunnel in Moran cars, a hopper-type car which kept the concrete agitated by rotation of the car itself. The concrete was pumped from these cars onto a beltcrete system which conveyed the concrete to its final location.

The arch concrete was placed from the north portal using the same batch plant. The movable arch forms consisted of 17 sections, each 24 feet long. The forms were removed after 24 hours and reset. Moran cars were used again for concrete transportation. For the arch placement, the concrete was pumped through a slickline. The concrete was vibrated as it was placed. Initially the concrete was overly stiff. Placement was improved when the slump was increased to $3\frac{1}{2}$ to $4\frac{1}{2}$ inches and the advancing edge of the fresh concrete was allowed to move ahead on a long sloping pattern.

Total excavation was 205,000 cubic yards. Average overbreak was 3.5%. Total lining concrete was 72,000 cubic yards.

The spacing of the contact grouting holes was selected by the inspectors based on knowledge gained during the tunnel excavation. The average spacing of the grout pipes was 4 feet. Grouting materials were hauled into the tunnel by train and mixed at the site. Grouting equipment consisted of a high-speed mixer, an agitator, an auger-type grout pump, and a work jumbo. Grout return lines facilitated pressure control. The general procedure was to maintain the connection to a pair of holes until the back-pressure reached 30 pounds per square inch (psi). Contact grouting was tested by redrilling and regrouting selected areas and by water testing some of the holes.

Consolidation grouting was used in areas where large overbreak had occurred and where rock pressure had distorted the bracing. Consolidation grout holes were drilled to a normal depth of 30 feet and grouted with pressures up to 100 psi. In areas of large overbreak, holes were extended to 60 feet. Drilling and grouting followed a split-spacing sequence, with the rings of grout holes horizontally spaced at 40 feet. Consolidation grouting terminated when the closing holes took less than 30 sacks per grouted interval.

Tunnel No. 1 was consolidation-grouted in five areas, with a high intake of grout in only one of the areas. At the faulted area near the north portal, both the contact and consolidated grout intake was high. One hole took 1,379 sacks of cement after it was drilled to 60 feet. Both types of grouting were done from a special rubber-tired jumbo.

Rubber waterstops were specified for construction joints. However, the contractor proposed the use of a split-type polyvinyl chloride waterstop. This type of waterstop was not as effective as expected, and a 9-inch center bulb-type waterstop was substituted. Even with the bulb-type waterstop, inflow appeared at some joints. Leaking joints were sealed with 100% solid epoxy resin and grouted.

The epoxy grouting procedure was: (1) the concrete surface was carefully sanded, (2) holes ¼ by 3 inches were drilled 1 foot apart along the joint, (3) the cuttings were removed by air, (4) wooden plugs were placed in the holes before applying the polyester sealer over joints which were dry, and (5) if the joints were wet, the connection was enlarged to form a groove ¼ to ½ inches wide which was filled with a waterplug surface sealer. After drying, grout was injected into one hole at a time from invert to crown. After the grout reached the next hole, the preceding one was plugged to retain the grout. Test cores indicated this method was successful in filling rock pockets around the waterstop and in stopping leaks.

Cast-in-place concrete pipe was placed between the north portal and the surge tank for a distance of 68 feet in a trench excavated by a dozer. Compacted backfill was obtained from the portal excavation and extended 10 feet above the pipe.

On completion of Tunnels Nos. 1 and 2, the excavation was made for Siphon No. 1 connecting the tunnels. Bedding materials was placed in the trench and compacted with whackers. The excavation then was backfilled to grade with concrete. Compacted earth was placed at the sides of the backfill concrete.

The siphon was cast in nine 24-foot sections and one 26-foot closure section at the portal of Tunnel No. 2. A manhole cast in the siphon adjacent to the dissipator and valve control vault provides access to the siphon and to the closure side of the first butterfly valve in the vault.

Backfill for the siphon was obtained from a borrow area near the south portal of Tunnel No. 2. The backfill was placed in lifts of 6 to 10 inches and compacted with whackers, pogo tampers, and sheepsfoot and vibrating rollers. After the backfill had been placed to a depth of 10 feet over the pipe, the drainage channel across the backfill was constructed. Grouted layers of stone were placed at both ends of the channel.

The south portal of Tunnel No. 2 was excavated by dozers on ¼:1 slopes with two 15-foot-wide benches. No ground water was encountered.

Tunnel No. 2 was driven for the entire length of 2,810 feet from the north portal. Rock generally was good and consisted of diorite gneiss for the entire length. The rock was slightly to locally moderately weathered, hard, strong, closely jointed to moderately blocky and seamy. No significant faulting or wide seams were encountered.

The tunnel was driven full face in a horseshoe section with the same type of steel supports and rock bolts that were used in Tunnel No. 1. Spacing of the sets generally was 4 feet center to center. Some areas were strong enough to stand unsupported or required only rock bolts. Shotcrete was not used in this tunnel except as auxiliary support for a 5-foot-wide fault near the south portal. Overbreak was minimal, rarely exceeding 2%. A typical round consisted of 125 10-foot holes. Gelex No. 2 was used. The powder factor averaged 2.5 pounds per ton of rock. A total of 72,000 cubic yards of rock was excavated. Ninety-eight working days were required to drive the tunnel, the shortest of the four tunnels in the crossing.

Four rock bolts in Tunnel No. 2 were selected for pull-out tests. The rock bolts were torqued to 350 foot-pounds. The expansion shell anchor at this torque was designed for a tension of 22,000 pounds. Tensile forces up to 30,000 pounds were sustained without yield of the bolts. Rock bolts were spaced on a pattern 3.5 to 5 feet apart. Occasionally, random bolts were used to pin a loose block of rock.

Ground water flow was even less than for Tunnel No. 1. The maximum inflow was 3 gpm at the north portal. The maximum cover was 618 feet near the mid-length of the tunnel.

When Tunnel No. 2 holed through, tension cracks appeared in the berm above the south portal and in the vertical face above the tunnel crown. These cracks resulted from stress relief along the joint planes associated with the fault which crossed the tunnel portal. The specifications required that the portal collar be in contact with undisturbed material. To meet these requirements, the contractor extensively buttressed the portal area. Steel channel walers were secured to the face area above the tunnel crown by 30-foot-long, grouted, rock bolts. An umbrella structure, just outside the portal face, was constructed using steel horseshoe reinforcing sets. Six- by six-inch cribbing was built up from these sets to the steel walers above the sets. Nearly 100 cubic yards of shotcrete was applied to the overbreak area above the portal, the first bench, and for 40 feet up the slope of the next incline. In addition, 770 linear feet of rock anchors were grouted into holes ranging from 20 to 40 feet in depth which secured 10-inch steel walers to the cut slope above the first berm. Compacted backfill placed against the vertical portal face completed the buttressing effort.

Contact grouting followed the same procedures as for Tunnel No. 1, except that because of the shorter tunnel length, the grout was mixed at the north portal and pumped to the grouting locations. Several exploratory holes were drilled where the tunnel lining at the crown was cracked or leaking. All holes were tight and encountered sound concrete and hard rock; therefore, consolidation grouting was not required. The faulted area near the south portal, however, was consolidation-grouted with 441 sacks of cement being used.

Tunnel lining for both invert and arch proceeded from the north portal using the same methods as were used for Tunnel No. 1. A total of 25,000 cubic yards of concrete was placed.

The 300 feet of cast-in-place pipe between the south portal and the transition to Pastoria Siphon was placed in the same manner as Siphon No. 1. The trench was excavated in the North Garlock fault. The foundation rock was considerably weathered, decomposed, and fractured. Suitable foundation for the pipe was obtained by overexcavating the entire area 5 feet and backfilling with compacted material. The backfill material consisted mostly of silty sand with some sandy clay obtained from an adjacent borrow area.

The canyon crossed by Pastoria Siphon is a topographic expression of the North Garlock fault. The Siphon was constructed in two phases. The first phase consisted of foundation preparation, embankment placement, and drainage across the embankment for Pastoria Creek. This was accomplished under the contract for Tunnels Nos. 1, 2, and 3. Phase 2 construction of the Siphon was by separate contract scheduled after embankment settlement.

The embankment on which the Siphon was placed consisted of two sections: a section upstream from the siphon alignment of approximately 400,000 cubic yards compacted to 90% and founded on existing ground, and the main embankment of over 700,000 cubic yards underlying and downslope from the siphon alignment compacted to 95% and founded on bedrock.

Excavation to bedrock allowed ground water to seep from the upstream sands and gravels. A 20-foot-wide bed of sand filter material 2 feet deep was placed on the bedrock and along Pastoria Creek.

The downstream embankment was placed in 6- to 12-inch lifts, moisture-conditioned, and compacted to 95% relative compaction. The dry density ranged from 122.1 to 130.3 pounds per cubic foot.

The upstream embankment was placed in a similar fashion but compacted to 90% relative compaction. During embankment placement, up to 10 gpm of ground water flowed through the foundation filter drainage system.

The grades for the Siphon on the canyon slopes were excavated by rippers and scrapers to a maximum cut of 40 feet with 1½:1 side slopes. The resulting foundation surface consisted of faulted and distorted rock. The siphon grades initially were used as haul roads during embankment construction. A total of 25,000 cubic yards of excavation was required.

Tunnel No. 3 (5,709 feet long) lies between the two branches of the Garlock fault. The northernmost 325 feet of Tunnel No. 3 is within a fault block of diorite gneiss in the North Garlock fault. The remainder of the tunnel penetrates the Pelona schist which is composed primarily of quartz-mica schist but includes graphite schist and quartzite. The formation has a well-defined foliation with a northeast trend of from 35 to 55 degrees. The trend of major joint sets is parallel to the tunnel alignment, or normal to the foliation. The schist is moderately hard when fresh, but it loses strength and becomes friable to soft when weathered.

The north portal of Tunnel No. 3 was excavated without blasting, in the same configuration as the other north tunnel portals. This portal cut required 267,500 cubic yards of excavation and four berms above the portal. Six berms were required on the slope just west of the portal.

The contractor requested and received authorization to reinforce the portal area temporarily rather than to construct the permanent buttress and headwall structure specified. The contractor assumed all responsibility for this change. As soon as the tunnel was faced in, the contractor reinforced the portal area. Six-inch channel steel walers and wire mesh were secured in nine rows with 40-foot-long, grouted, rock bolts. This protection extended up to the first berm. Fourteen 20-foot and eight 40-foot No. 18 rebars were grouted in an arch pattern 1 foot outside of the tunnel "B" line. An umbrella structure using five horseshoe ribs on 4-foot centers then was erected. The vertical face area and the lower two berms and rock slopes were sprayed with an asphaltic emulsion for resistance to slaking.

Six weeks after completion of the portal reinforcement and following heavy rains, a 1,300-cubic-yard slide occurred in sheared diorite gneiss immediately west of the portal (Figure 191). No corrective action

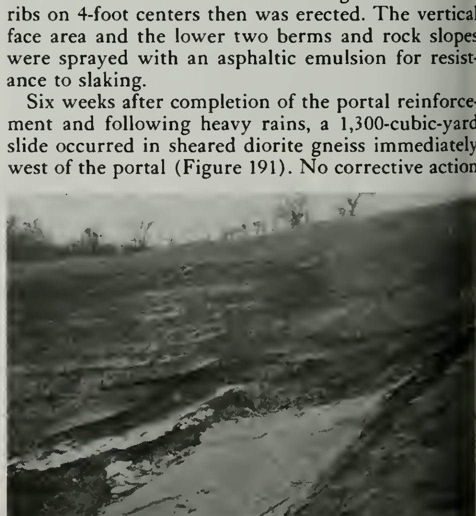

Figure 191. Portal Slide—Tunnel No. 3

was taken, and the slide enlarged and moved downward as the contractor removed rock from the toe of the slide that impeded construction traffic. Six months after the slide occurred, wire mesh and a concrete slurry were applied to the slide. Two years later, a concrete slurry was sprayed into the tension cracks which had opened upon the lowest berm. This slurry apparently lubricated the slide block which moved into the tunnel distorting the steel rib sets as much as 5 inches.

The rock structure in Tunnel No. 3 ranged from moderately jointed to completely crushed but mostly was moderately blocky and seamy. Eleven shears greater than 5 feet wide, 43 between 3 inches and 5 feet wide, and 19 less than 3 inches wide were encountered or an average of one shear per 78 feet.

When the rock fracture planes containing clay were exposed to water, the blocks of rock tended to squeeze out along those planes. Ground water would weaken soft plastic materials in areas of breccia and intensely altered rock, and the accompanying seepage pressures caused the ground to squeeze.

Poor tunneling conditions in Tunnel No. 3 required different driving techniques. A full-face excavation was used only 57% of the time, mostly in the northern half of the tunnel. A top heading was used for 32% of the time with the remaining 11% of the driving accomplished by the heading and bench method. Two separate jumbos joined together were used. The upper, or "Gantry", jumbo was mounted on rails suspended at the springline. The lower jumbo was mounted on a crawler tractor. The use of separate jumbos facilitated the use of the top heading and bench method of tunneling. A total of 141,000 cubic yards of rock was excavated.

Overbreak for Tunnel No. 3 was less than for Tunnel No. 1, averaging 3%. The support system varied from four-piece, M8X40 (8M40), horseshoe sets to M8X40 six-piece sets. In the Garlock fault at the southerly end of the tunnel, M8X58 sets were used. Set spacing varied from 6 feet to the specified 12-inch minimum. Double sets, which placed the flanges closer than this minimum, were used occasionally. The auxiliary set was removed prior to lining.

Rock anchors also were used occasionally. This procedure consisted of drilling a 2-inch-diameter hole 8 to 20 feet deep and inserting a grout-filled cylinder of hardware cloth. A piece of No. 11 reinforcing bar threaded on one end then was driven into the hole and, when the grout had set, the bar was torqued to 350 pounds. These anchors were used to pin the arch sections of a set when the top heading method was used. This supported the ribs while the bottom heading was being mined. These grout anchors also were used to pin the post section of a set instead of using invert struts.

Walers of steel channel running longitudinally across several sets also were pinned to the tunnel rock with grout anchors to resist squeezing pressures. Concrete and chemical grouting was employed ahead of the face for stabilizing the ground. Shotcrete was used to help support the sets and lagging. In one instance, 30-pound, interlock, sheet piling was driven above the crown for 25 feet before advancing the heading.

When the tunnel had advanced 3,000 feet from the north portal and progress had slowed to 42 feet in six weeks, the contractor decided to open up a heading at the south portal. A 7-foot by 7-foot crown drift, extending 12 inches above the "B" line, was started. This drift was supported by 12- by 12-inch timber posts and caps and extended to the slowly advancing north face. This crown drift allowed ground water which had slowed the advance from the north to drain.

After holing through the crown drift, top headings were advanced from both portals followed by bottom headings. Some remining was required to achieve the specified alignment.

Concreting was similar to Tunnels Nos. 1 and 2. Invert concrete was placed southward from the batch plant which had been moved from Siphon No. 1 to the vicinity of the north portal of Tunnel No. 3. Arch concrete was placed northward from the south portal using the same batch plant and transporting the arch concrete by train on rails placed on the invert concrete. A total of 51,000 cubic yards of lining concrete was placed.

Ground water was not a severe problem. Average flow was about 80 gpm with the maximum being 200 gpm. Springs in Sal Cito Canyon near Tunnel No. 3 appeared to be drying up, and correlation of the decreased spring flows with the tunnel driving was investigated. The Canyon, the head of which lies above the tunnel alignment, trends southeasterly away from the tunnel alignment. The springs, 1,200 feet to the southeast, and the tunnel invert are at about the same elevation.

A fault, which trends along the Sal Cito Canyon, acts as a barrier to the downslope movement of local ground water. This fault separates gneiss from schists, and the fractured rocks also act as ground water reservoirs. The water, unable to flow beyond the fault, rises to the topographic low of the fault and occurs as springs in the Canyon.

The fractured rocks are hydraulically connected to those encountered in the tunnel below and, when penetrated, reduced the hydraulic head on the springs and therefore the volume of flow. Since the amount of water seeping into the tunnel was not severe, it was expected tunnel lining and pressure grouting would cut off this release of water and the springs would return to their preconstruction flow. This evaluation subsequently was found to be correct.

The maximum cover on the tunnel was 643 feet.

Pastoria Siphon

Design. Pastoria Siphon (Figures 192, 193, and 194) was designed assuming a Manning's "n" of 0.012 for a maximum hydraulic gradeline elevation of 3,200

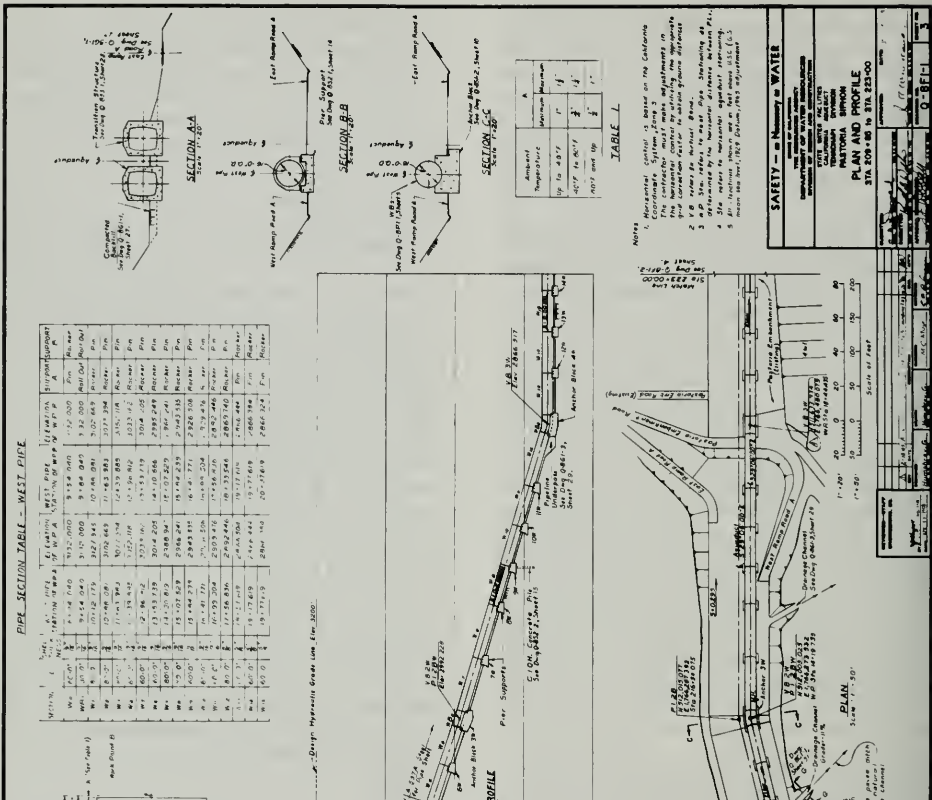

Figure 192. Pastoria Siphon—Plan and Profile

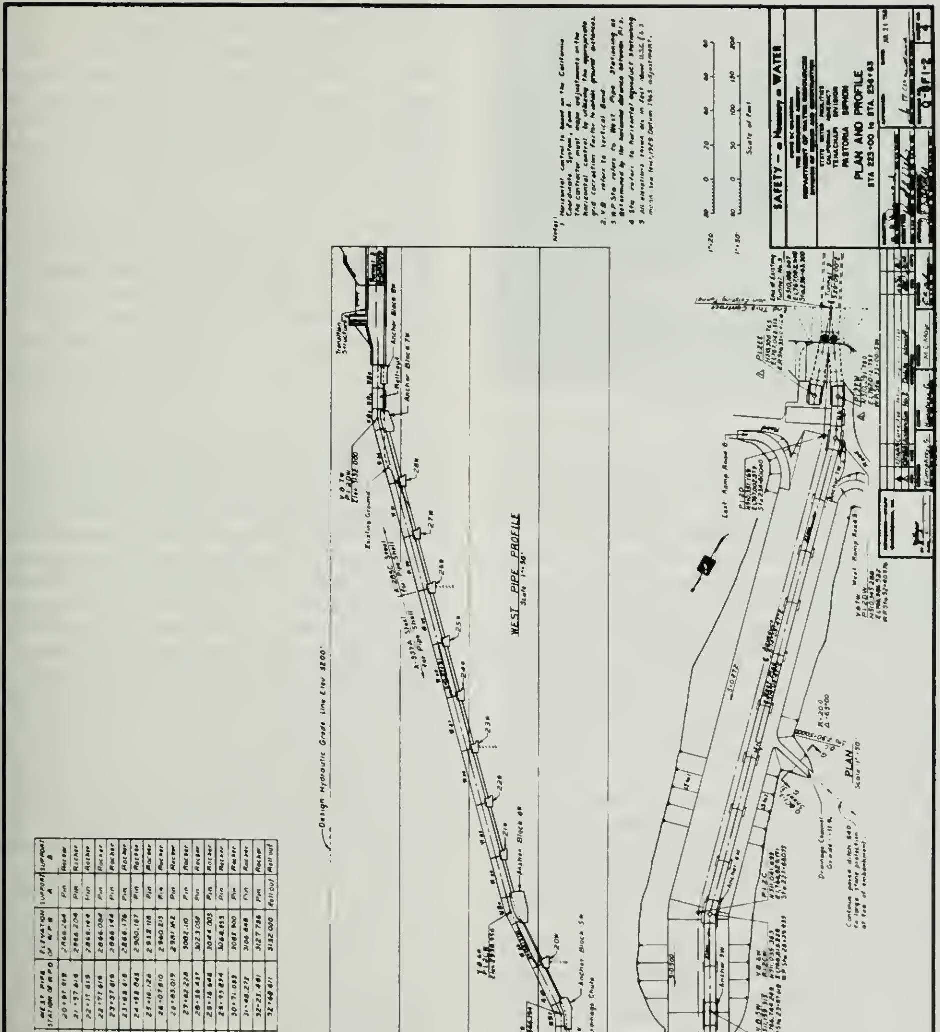

Figure 193. Pastoria Siphon—Plan and Profile (Continued)

Figure 194. View of Pastoria Siphon

feet. At that elevation, the maximum capacity will be 5,360 cfs when the second 16-foot barrel and extensions of the northern gate shafts are added. Presently, the one existing barrel can carry the 4,095-cfs design discharge of A. D. Edmonston Pumping Plant without overtopping the present gate shaft.

The plans and specifications were prepared for either a double-barreled siphon (Schedule 2) or a single-barreled siphon (Schedule 1) having transition sections for accommodating future installation of the second barrel. After receiving bids on both schedules, an economic analysis was made considering present and future construction costs, the time of need for the increased capacity, and the construction funds available at the time. The resulting decision awarded the contract on Schedule 1, thus deferring the second barrel.

The Siphon consists of bifurcated, reinforced-concrete, transition structures; rectangular air-shaft sections; a 16-foot-inside-diameter, steel, roll-out section; and one steel barrel. The length of the transition from tunnel section to siphon barrel is 109 feet. The transition sections for the future barrel are presently bulkheaded at the air shafts.

The air shafts serve three purposes: (1) provide siphon access, (2) accommodate bulkheads for isolating a barrel, and (3) function as air vents. The shafts have provisions for increasing their vertical height.

The 30-foot-long roll-out section travels on wheels which roll on a track supported by a concrete roll-out pad. The wheels normally are removed. The section weighs 18 tons empty.

The individual sections of Pastoria Siphon are connected by bolted sleeve-type couplings and are supported by ring girders on concrete foundation pedestals through pin-connected rockers. Bend sections are strengthened with stiffener rings. Seven reinforced-concrete anchor blocks resist hydraulic thrust forces.

The upper sections of the pipe were fabricated of A285C steel having a working stress of 15,000 psi. The higher stressed lower sections are A537A steel with a working stress of 25,000 psi. All stiffener rings are made of the higher strength steel. Wall thickness of the pipe was determined from combined beam stress and hoop tension. The rockers and pins are of A36 steel and are similar in design to rocker bridge assemblies. The pier supports basically are short columns on spread footings. Five of the pier footings are supported on drilled-hole cast-in-place piles.

The independent footings provide for individual settlement or movement, and the mechanical couplings between pipe sections provide for changes in lengths caused by temperature variations and provide some flexibility in the event of seismic disturbance.

All structures were designed for a 0.2g seismic force. Manholes are spaced at 500-foot intervals along the siphon barrel. A 6-inch steel line with a 6-inch plug valve at the low point in the barrel drains the Siphon into Pastoria Creek.

Construction. The north transition structure was constructed first and is founded on faulted and sheared diorite gneiss. The south transition structure is founded on gneiss of the Pastoria thrust plate. Because the north and south transitions are structurally similar, many of the concrete forms were reused for the south structure. The transitions were constructed in four sections: the invert, the walls, the top, and the air shaft. The anchor blocks and pier supports for the siphon barrel were not constructed. Five piers were set on cast-in-drill-hole piles for increased shear strength and resistance to uplift. The holes for the piles were drilled with a 30-inch bucket auger to a depth of about 25 feet and belled out at the bottom to 48 inches in diameter.

The pier pedestals were cast and the pipe sections then placed and secured by the pin connection between the ring girders and the pedestals. A total of 6,000 cubic yards of structural concrete and 3,000 cubic yards of anchor block concrete were used.

The pipe was fabricated near Castaic, California. The pipe sections, up to 80 feet in length, were too large for transport on the regular highway system. However, the use of the old "Ridge Route" and a portion of a Tejon Ranch road was authorized. Both roads required improvement for this purpose.

The bend sections at the transitions were placed first, followed by the horizontal sections on the embankment. The north and south legs were placed last. A special, elevated, wedge-shaped, ramp structure was required for crane access in placing the steep leg sections. The pipe sections were delivered with one coupling bolt ring on one end and one bolt ring plus the sleeving on the other end. Two gaskets then were installed on both sides of the sleeve. The bolts were placed, in alternating directions, through the two bolt rings and tightened with the gasket in position between the sleeve bevel and the outside diameter of the pipe section.

Ground water draining from the north portal of Tunnel No. 3 was utilized for the hydraulic test of the line. Thirty days were required to fill the Siphon. During filling, if any part of a joint leaked, the bolts 12 to 18 inches on each side of the leak were tightened until the leak stopped. All leakage was contained by the time the water reached the top of the air shaft. On completion of the hydraulic test, compacted backfill was placed.

The Pastoria access road was relocated to the south side of the embankment, and asphalt-surfaced maintenance roads were constructed along each side of the siphon barrel. Two steel foot bridges were erected across the concrete drainage channel of Pastoria Creek. One of the foot bridges will serve the future siphon barrel.

Beartrap Access Structure

Beartrap Access Structure (Figures 195 and 196) connects the south portal of Tunnel No. 3 with the north portal of the Carley V. Porter Tunnel. The contractor for the Phase I work for Carley V. Porter Tunnel performed most of the excavation for the access structure. The Phase II contractor, while developing the portals of the adjacent tunnels, completed the excavation except for minor grading and compaction of the subgrade.

The structure is a 315-foot-long section of reinforced concrete. The interior diameter over a distance of 48 feet varies from 23.5 feet at Tunnel No. 3 to 20 feet at Carley V. Porter Tunnel. The minimum wall thickness is 2 feet - 6 inches. The exterior cross section is shown on Figure 195.

The portion of the transition section within the protective portal for Tunnel No. 3 joins the tunnel lining and was cast against the steel sets of the portal protection structure. The remainder of the structure was cast in five 50-foot sections and one 17-foot closure section at the portal of Carley V. Porter Tunnel. Nine-inch center bulb waterstops were placed in the joints between sections.

A 36-inch-inside-diameter steel-pipe manhole with a steel blind flange provides access. An air shaft near the north portal of Carley V. Porter Tunnel vents this section of the crossing.

Surface runoff and drainage down Beartrap Canyon is passed over and around the tunnel access structure by one 72-inch and one 75-inch, corrugated-metal, pipe culverts.

A 30-inch steel-pipe stub was installed near the manhole for a future Tejon Ranch turnout.

Six feet of compacted backfill was placed around and over the structure.

Carley V. Porter Tunnel—Phase I

The last major exploration prior to driving the Carley V. Porter Tunnel was a 3,600-foot-long, 5- by 8-foot, pilot tunnel. The Tunnel was driven from the north portal on the tunnel alignment and served as a crown drift during final excavation. This pilot bore provided specific information to both designers and prospective bidders on geologic conditions in the Garlock fault and the Tejon Lookout granite south of the fault.

The north portal of the Tunnel is on the southeastern side of Beartrap Canyon. This canyon is a narrow, steep-walled, heavily wooded, topographic feature of the main Garlock fault. Pastoria Creek which also flows down this canyon is the main drainage feature of the central Tehachapis and normally is dry in the summer. Winter flows, however, required diversion by two corrugated-metal pipes, as previously mentioned, around the construction site and the permanent facilities.

The major underlying rock in Beartrap Canyon is the Pelona schist, heavily fractured and altered by movement on the Garlock fault. The layers of alluvium overlying the bedrock are moderately well-bedded but contain skip-graded lenticular gravels and coarse to fine sands. The maximum depth of these deposits is less than 25 feet. Large boulders, 5 to 6 feet in diameter, occur throughout these deposits but are more common in the lower portions.

The Pastoria Creek diversion consisted of three units: (1) a cutoff wall to bedrock; (2) a catch basin formed by an earthen dike over the cutoff wall; and (3) parallel 72- and 75-inch corrugated-metal pipes from the basin to the north side of the Canyon discharging below the spoil area back into Pastoria Creek.

The contractor, after clearing and grubbing, sublet construction of the cutoff wall. Specifications for the wall permitted a choice between (1) a mixed-in-place concrete wall; or (2) a steel sheet-piling wall. Alternate (1) was used and the subcontractor drilled overlapping 16-inch-diameter holes using a cement slurry and drill cuttings for aggregate. The drill bits wore rapidly in the rocky ground and, because boulders were encountered, the wall never penetrated below 19 feet, about half of what was anticipated.

Accordingly, the wall did not cut off the underground flow. At first a grout curtain was planned. Subsequent excavation did not encounter much water and then it was decided that the flow could be controlled in the work areas by drainage ditches and short sections of small corrugated-metal pipe.

The intake for the main diversion conduits was formed with pipe elbows and welded gratings. The conduits were laid in trenches and backfilled with crushed rock. Wire tie-downs in grouted holes bored on each side of the pipe anchor the pipe.

Carley V. Porter Tunnel—Phase II

Design. This tunnel (25,075 feet long) of the Tehachapi crossing was designed as a low-pressure (less than 60 feet of head) conduit with an inside diameter of 20 feet and a minimum, unreinforced-concrete, lining thickness of 10 inches.

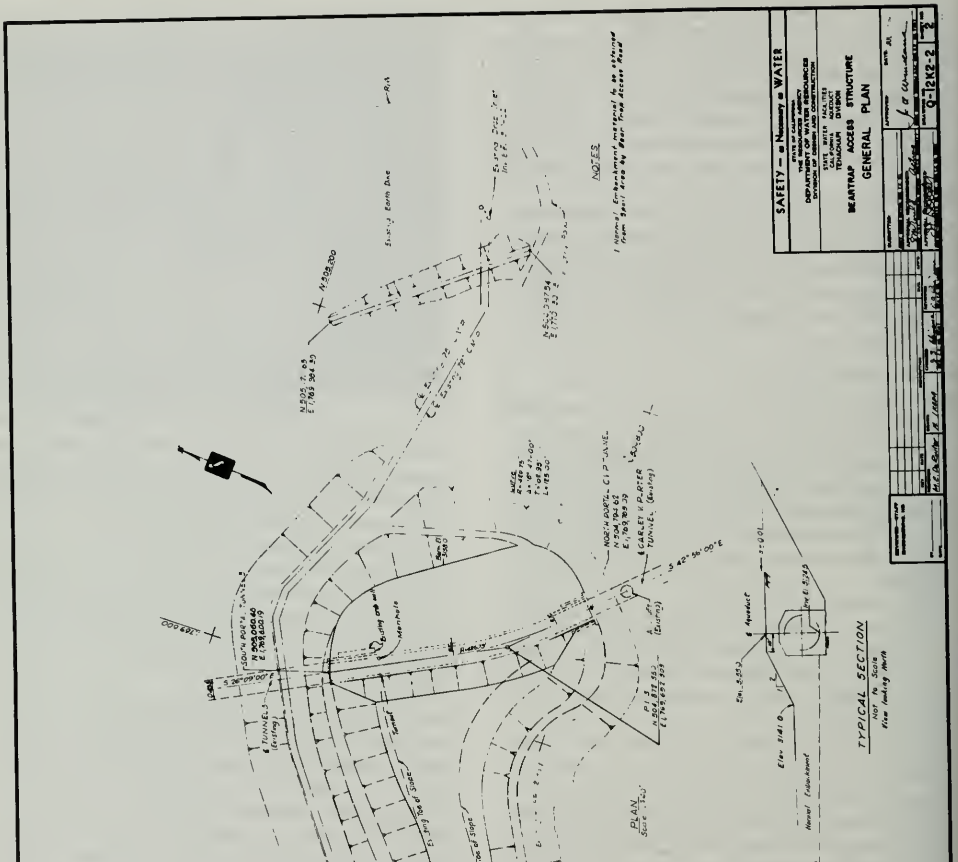

Figure 195. Beartrap Access Structure—General Plan

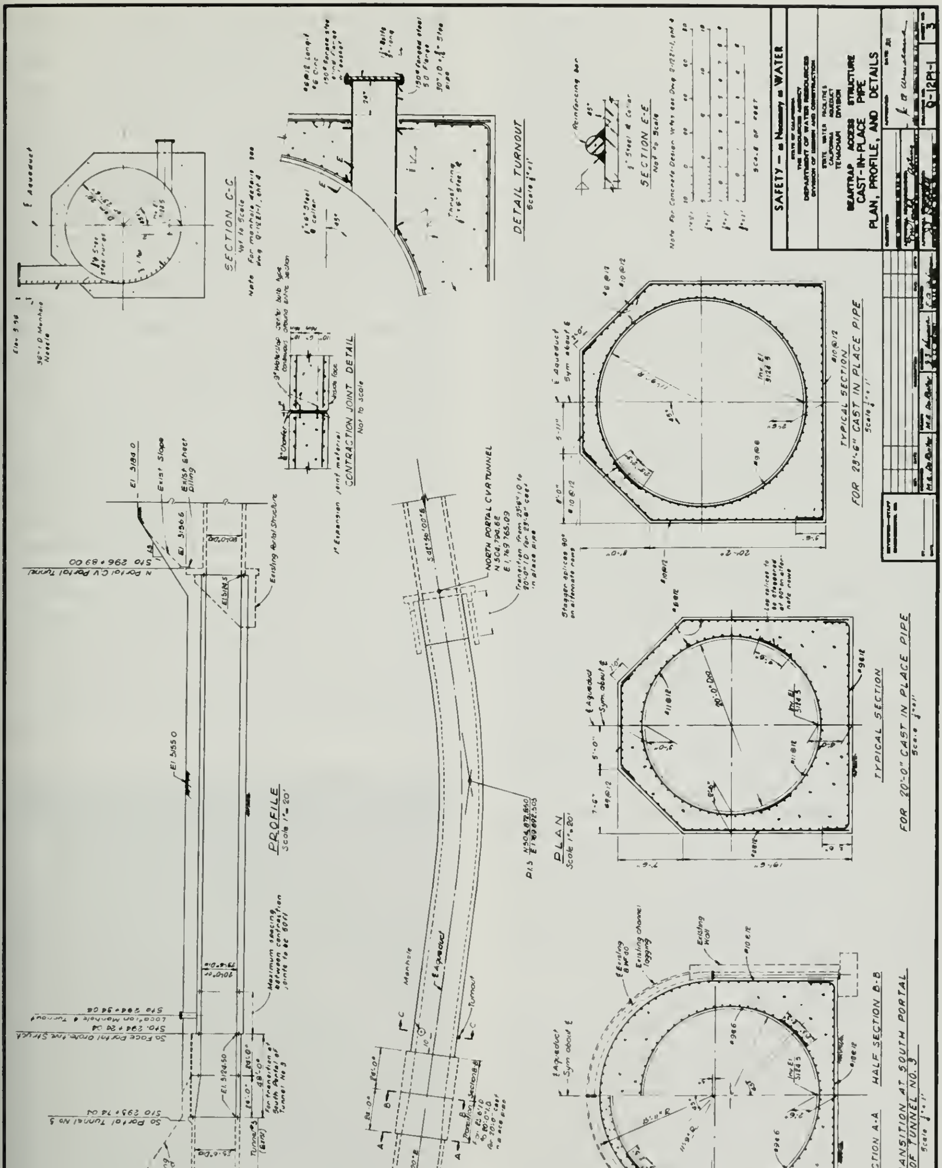

Figure 196. Cast-in-Place Pipe—Beartrap Access Structure

The method of tunnel support was not specified. Anticipated alternatives were (1) a circular rib system, (2) a horseshoe rib system with or without an invert strut, and (3) a post and rib system with semicircular ribs for the upper half. The horseshoe rib system was considered the most likely choice by the contractor. Maximum spacing of sets was established at 4-foot centers with the minimum spacing at 12 inches between flanges. The bracing was specified as 8-inch-deep ribs at 40 pounds per foot. Even with the poor ground known to exist for much of the tunnel length, it was expected this support system would be adequate providing the circular system was used in the Garlock fault area near the north portal. If heavier supports were required, heavier 8-inch ribs were to be used and the change in steel-support quantities would be negotiated with the contractor. A maximum set deflection of 3 inches was permissible. Replacement was required for distortions greater than 3 inches.

Buttressed-wall portal structures were specified for both portals supplemented by bin-type retaining walls at the south portal. Load cells were planned at specified locations. The connection of the load cells to the support system was the responsibility of the contractor.

The south portal excavation was performed under a separate contract which also included provisions for relocating the drainage of Cottonwood Creek around the site.

Construction. The south portal and the initial 100 feet of Tunnel No. 3 were constructed by the contractor for the second phase of the Carley V. Porter Tunnel. This was done so that the Tunnel No. 3 contractor could drive from both the north and south portals without interfering with the concurrent work on the Carley V. Porter Tunnel.

Except for the vertical face, the south portal excavation (Tunnel No. 3) was made on a 1½:1 slope with 10-foot benches at 30-foot vertical intervals. Four benches were required, and the lowest bench was made 30 feet wide for relocation of the existing corrugated-metal pipes used for draining Beartrap Canyon. No reinforcement or portal protection was used. The rock structure was a jumbled mass of fault blocks separated by clay gouge.

This first 100 feet of tunnel (Tunnel No. 3) was driven from the south portal with multiple drifts. Small side drifts at invert elevation were excavated and an 8-inch, wide-flanged, steel beam laid as a wall plate for the horseshoe sets which were placed after hand or air-spade excavation of a top heading. On completion of the top heading, the remaining ground between the pilot headings and the top heading was excavated. Invert struts and posts were used to support the lower heading. The sets, which were of M8X58 steel, were heavier than the M8X40 sets used in Tunnels Nos. 1 and 2. They were placed on 1- to 3-foot centers with most on 2-foot centers. Six-inch steel channels were used for lagging between sets (Figure 197).

Upon completing this section, the tunnel face was still in the Garlock fault, and a temporary wooden bulkhead was erected over the heading face.

Two days after completion of the top heading, the east side of the portal face collapsed. A metal-bin retaining wall was installed to prevent further sliding. Two months later, to buttress the portal further, an umbrella and compacted backfill were added.

The Carley V. Porter Tunnel was driven from both portals simultaneously through three distinctive types of rock formation. The first several hundred feet of tunnel from the north portal are in the Garlock fault. The first 3,000 feet from the south portal are in geologically recent lake deposits. Two down-faulted blocks of severely weathered granite were encountered in those sediments. The remaining 85% of the tunnel length was in the Tejon Lookout granite.

Cottonwood Creek near the south portal of the Tunnel flows largely underground after reaching the valley floor. The tunnel alignment to the north underlies Cottonwood Creek for several thousand feet. A dike approximately 500 feet long and 25 feet high was constructed northeast of the south tunnel portal to divert flow around the portal area. In stripping the dike foundation, underground flow was encountered at a depth of 3 feet which hampered the stripping and a positive cutoff was not obtained. Because of this some flow seeped out above the tunnel portal during construction.

The portal cut was mostly in lake deposits with some older alluvium and more recent fan deposits. Because of the slaking and erodible nature of the

Figure 197. Tunnel No. 3 Under Construction

deposits, the slopes were laid back to 3:1 with 20-foot berms at 20-foot vertical intervals. A net of corrugated-metal downdrains was used to drain the slopes. The area was first stripped to a depth of 5 to 10 feet by tractors and scrapers. Some ripping was required. A maximum of six berms was required on the canyon slope above the south tunnel portal. Little ground water was encountered.

The portal face was excavated by the tunnel contractor using air spades and occasional light powder charges. The face was protected by horizontal walers connected to vertical channels pinned to the vertical face with rock bolts.

The north portal excavation deepened the existing Phase I cut by 10 feet. The tunnel invert was below Beartrap Canyon, and a small sump at the portal face was used to pump inflowing water to the inlet for the 72- and 75-inch pipes already in place. At the north portal, sheet piling was used to support the cut adjacent to the portal face.

The headings were driven with forepoling shields (Figure 198). Two identical shields were constructed, one for each heading. The support system was made up of 4-foot-long circular sections assembled within the shield forming a continuous steel system.

Each forepoling shield was made up of two units: (1) the forepoling plates which were a semicircular series of nine overlapping toothed plates which, when extended, protruded 5 feet ahead and around the top half of the shield. These forepoling plates were separately and individually extended by nine hydraulic jacks; and (2) the shield which was constructed of welded T-1 steel members on a 24-foot - 4-inch outside diameter.

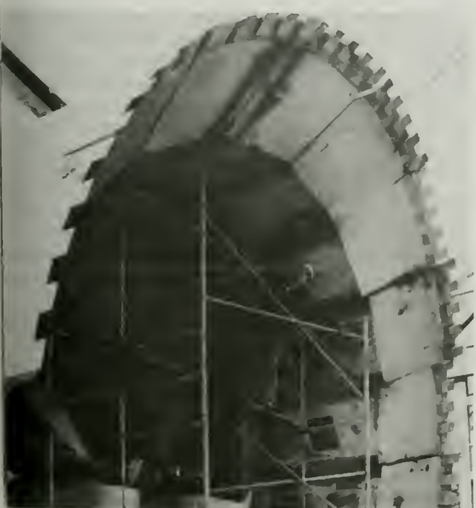

Figure 198. Forepoling Shield

The crown length of the shield was 24 feet and the invert length 15 feet. The interior of the shield was divided into two working sections. The frontal section contained the control panel, the nine poling plate jacks, the 23 independently operated jacks for advancing the shield, and four to six jacks for breastboarding the face. This section also contained a hydraulically operated lift and placement mechanism for the supporting liner-plate segments. The posterior section of the shield was used for the assembly and placement of the support system.

The total weight of the shield was 325,000 pounds. The breastboard jacks were 100-ton capacity. The other jacks were 200-ton capacity. The hydraulic system was designed for 10,000 psi maximum pressure but pressures up to 13,100 psi were used.

The support system consisted of angle-iron-stiffened, $\frac{1}{4}$- to $\frac{5}{16}$-inch-thick, steel-plate, skin sections. Each 4-foot circular section consisted of one invert segment of 108 degrees, four side sections of 60 degrees each, and one finishing or crown section of 12 degrees. The skinplate sections were stiffened with two circumferential rings of 8-inch angles, located at each end of the 4-foot section and cross-braced longitudinally with 4-inch angles. Butt plates equal in width to the 8-inch angles and the liner plate closed the two ends of each segment. The segments were butt-bolted together by four bolts. The 4-foot rings were bolted together with 48 equally spaced peripheral bolts. During assembly, the segment joints were staggered about 15 inches.

Three weights of sets were used: lightweight, 10,000 pounds per set; heavyweight, 12,200 pounds per set; and extra heavy, 13,700 pounds per set. All three types had an outside diameter of 24 feet, which was 4 inches less than the bore diameter of the shield. In requesting approval to use this support system, the contractor specified he would fill this annular space with pea gravel or grout. This requirement, except for brief periods, was not observed during the tunnel driving period. Access to the walls of the bore behind the liner plate sections was impossible without cutting a window through the plate. Therefore, normal blocking of the support system was not possible with this shield system. All sets were 4 feet in length. Early in the contract, the contractor experimented with four sectioned, 5-foot-wide, flanged, support rings with channel-iron lagging but discontinued this method due to time and erection problems.

The normal driving cycle from both headings was: (1) drill and blast (a minimum heading advance of 4 feet - 2 inches was required to place a supporting set); (2) muck out most of the round; (3) advance the shield by pushing on the last previously erected set. As the muck sheet from which the muck loading equipment worked was above the tunnel invert, all of the muck could not be removed before advancing the shield; (4) erect within the shield the six segmented support sets; and (5) push out the poling plates and breastboard jacks if needed to retain the face and start a new cycle. At times, steps 4 and 5 were reversed in sequence depending on rock conditions.

Because the support set rested on the shield during erection, the support set would drop downward when the shield was moved forward and out from under the set. If allowance was not made for this drop during initial bolting in the the set erection process, torsional stresses were imposed on the sets.

In moving the shield forward, the following resistances were overcome: (1) the friction of the ground on the exterior surface of the shield, (2) the friction of the lining set erected in the tail of the shield, and (3) the displacement of the ground in front of the shield which had not been removed during the mucking cycle.

A laser beam shining on a target at the tunnel face was used for maintaining line and grade. It was very difficult to keep the shield on line and grade because of frequent changes in rock conditions and the accompanying pressure changes on the shield. Soft zones caused the shield to nose down. Once started down, alignment was difficult to correct. If the lifters (bottom holes) did not break the rock in the lower part of the face sufficiently, rock points hidden in the remaining muck had a tendency to nose the shield upward. The shield became stuck on many occasions when attempting to regain line and grade.

Since there was no way of reversing the shield when it became stuck, hand methods were required to release the shield. Small drifts were driven outside of the shield and the invert cutting edge was uncovered by hand excavation.

The number of holes required for a round varied with rock conditions. Generally, few holes were required in the upper face. Because the drill machines could not operate close to the sides of the tunnel due to the jacking equipment, and because the lower holes were drilled from the muck sheet, all outside holes had to be angled sharply toward the tunnel periphery. These angled holes, if deep enough for set clearance, contributed to overbreak and, if not deep enough, would often result in protruding ribs of rock.

At the south portal heading, the initial excavation was made with a Memco mining machine. This machine had a fully articulated, bucket-type, digger head. In soft ground, it gouged out the face, dragging and dumping the broken rock onto an inclined elevator. The rock then was loaded into muck cars by a gantry-mounted belt conveyor. Downtime for repairs, mainly to the hydraulic system and the inclined elevator, amounted to 50% of the total work time. Use of this machine was discontinued after 800 feet of tunnel had been driven. Blasting and mucking methods then were used for the remainder of the mining from the south portal.

The tunnel was holed through after 1,288 days for an average advance of 19.5 feet per day based on three shifts, five days a week initially and six days a week subsequently. At times, no advance occurred for several days. During several periods, downtime lasted several months at both headings. These longer periods occurred at the north heading for the construction of a new shield and the south heading while remining a cave-in.

When required, additional support was provided to the liner plate sets by jump sets, horizontal bracing, knee braces, or a combination of horizontal and vertical braces. The jump sets usually were W8X40 (8WF40) steel ribs and were installed in the center of the regular panels. Some of the jump sets were installed full circle, while others were installed only to the invert segment butt plates. The extra supports were made up of welded W8X40 members.

After a lapse of about 22 months, the contractor began experimenting with gunite as reinforcement for sets showing distortion, using a mix of four parts sand and one part cement. At times, a dry accelerator was added. At first, attempts were made to completely fill the set sections to the tops of the stiffener rings with gunite. This attempt was unsuccessful because the thick coating of gunite would not adhere to the curved sections of the liner plate. The space between the liner plate and the flanges of the stiffener angles was filled, however.

The gunite provided some support but could not resist any bending stresses from distortion of the liner sets. The gunite between the flanges and the liner plate helped prevent the flanges from bending. Bending frequently occurred from high jacking stresses and tight bolting during initial assembly of the set.

Overbreak at the north heading, except in four instances where it reached or exceeded 100%, was nominal during the first 3,600 feet of advance. In the next 15,000 feet, 65 separate overbreak runs of over 100% were recorded. Major runs were estimated at 5,700, 4,060, 3,180, 2,600, 1,375, and 1,000 cubic yards, with several more in the 500- to 800-cubic-yard range.

The first extensive delay occurred in the south heading in March 1967, after an advance from the portal of 4,400 feet when the shield became stuck in weak, blocky, seamy granite. The tunnel face was caved for the full width of the tunnel, and water inflow at the face was about 5 gpm. At the time, activity was limited to driving the piling, breastboarding the face, and replacing segments of the tunnel supports damaged by jacking the shield. Water inflow increased to 25 gpm.

Cottonwood Creek is 240 feet above the tunnel grade at this location. The contractor diverted the surface flow of the creek and commenced side drift around the shield. The shield was advanced 3 feet again distorting the liner plates, inducing further caving which, by then, was approximately 600 cubic yards. On March 12, the caving extended to the surface about 40 feet east of the tunnel centerline, producing a crater about 25 feet in diameter and 10 feet in depth. The next day, another crater appeared 800 feet back toward the south portal. A run of several hundred cubic yards of muck had flowed into the tunnel when it had been previously driven underneath this location. A third area of surface subsidence occurred a month later; this one was 130 feet south of the area of the first subsidence.

Both surface and face grouting were undertaken in an attempt to stabilize the muck run. A total of 9,000 gallons of chemical grout was pumped into 18 holes in the heading. One large bore hole from the surface was grouted with 1,300 sacks of cement, 90 sacks of Calseal, and 450 pounds of calcium chloride. The shield was freed the week of March 31 and advanced 8 feet. During April, the shield was advanced 32 feet during periods of surface and underground grouting.

Deep-well pumps were installed in May in two large holes from the surface in an attempt to dewater the affected zone. However, water extraction was negligible. A well point system of 22 points 15 feet long installed from the surface produced only 25 gpm. The flow in Cottonwood Creek was much greater. During the week of May 12, conditions improved and the shield was advanced 48 feet. Normal operations were resumed the following week. Total advance had been 52 feet in 72 days. Neither the chemical or regular grouting effectively penetrated the clayey decomposed granite.

The heading from the north portal was delayed for 206 days in May 1968, when the shield was distorted beyond repair, and a new shield (without forepoling plates) was moved into place. The first shield drove 9,048 feet of tunnel which, considering the ground penetrated, was a remarkable achievement.

The shield became stuck in a severely weathered to completely decomposed granite laced with many thin, clay, gouge seams. Overbreak had been 700 cubic yards in the previous 16 feet of tunnel. Although water inflow at the face was less than 1 gpm, a surging flow of 30 to 75 gpm had been encountered early in a strong shear zone 125 feet farther back. That water probably moved along the liner plates to the face, softening the rock.

In freeing the shield, side drifts were started at springline but both drifts caved. A cavity 40 feet high was observed above the east drift before the drift was abandoned. The shield began to deform and move easterly, and an 8-inch gap opened up between the shield and the last set. The shield was horizontally reinforced with three 114-pound 18-inch by 12-inch I-beams and the deformation stopped.

The contractor commenced a 30- by 30-foot chamber ahead of the old shield to receive the new shield. Wall-plate drifts were started. Two of the three cross struts were cut, resulting in further distortion of the shield near the cutting edge. Four 14- by 14-inch 190-pound H-beams then were placed across the old shield for further cross bracing. Distortion stopped, but these heavy beams buckled severely in the process. The contractor then removed the inner portions of the shield and supported the shield skin with W8X58 and W10X66 circular ribs.

After the two wall-plate drifts and a crown drift were excavated, the chamber for the new shield was completed by a top heading and bench method. The new shield then was erected and tunnel advance resumed.

The major delay in the south heading occurred on December 20, 1967, when 64 feet of tunnel collapsed 1,900 feet in from the south portal while being remined to correct alignment as explained later. The heading was 3,842 feet ahead of the collapsed area. The collapse trapped 17 men at the tunnel face. Fortunately, no men were working at the time of the cave-in in the area which collapsed.

Communications and supplies to the trapped personnel were delivered through 8-inch and 4-inch pipelines, which had remained partially open. The men were rescued 22 hours after the collapse through a small tunnel driven through the collapsed section. Tunnel cover was about 140 feet. An escape drill hole had been started from the surface but was abandoned upon rescue of the men.

The collapse occurred in a large faulted area of lake deposits. The total breadth of the fault measured along the centerline of the tunnel was 428 feet. Besides claystone, the zone included fault breccia, plastic clay gouge, limey claystone, marl, silty sandstone, and limey sandstone.

During the initial tunneling leading into this fault, little dynamiting was required. Loosening with pop shots below the springline, trimming the crown with the forepoling plates, and excavating with the Conway mucker were the general procedures. No pea gravel or grout were placed in the annular space around the support system. The ground moved in against the liner plate during initial placement of the sets or shortly thereafter. In the fault, the ground became weaker. Sloughing and squeezing ground required breastboarding. The shield when advanced started downgrade. The contractor, in the invert section of the last set, had placed 6-inch by 6-inch timbers between the ribs to concentrate the push at the bottom of the shield and help direct it upward. This helped regain grade but two sets were distorted and had to be replaced. The shield was 1 foot below grade at this time.

The shield again moved downward 2 feet below grade in soft to stiff, moist, plastic ground and breastboarding was required. Although steel sleds were placed under the invert nose of the shield, the shield continued moving downward another foot. Additional sleds then were installed. As the ground improved, the shield moved upward to 1 foot above invert elevation but grade was regained within the next 30 feet. The sets established earlier in the fault crossing were taking weight; however, no visible distortion was observed prior to remining. During the year since the original mining, the squeezing ground had undoubtedly reached an equilibrium condition.

The decision was made to remine the area and bring the support rings back to proper grade. Since the tunnel heading was being worked at the same time, it was necesary to allow passage of traffic to and from the face during the remining. The existing sets were extended upward by means of "dutchmen", creating an hourglass shape for the remined sets. These hourglass sets were weaker than the circular section.

The "dutchmen" were welded steel sections which were placed in the existing support segments. These spacers lengthened the segment in which they were placed. In sets which required extensive raising, sufficient extension required spacers to be placed in the 60-degree springline sections, as well as the crown closure sections. During the remining, many of the peripheral bolts connecting the liner segments were removed to spread out the sectors of liner plate. Many of these bolts could not be replaced because of the final configuration of the elongated sets. This condition resulted in lessened ability of the sets to transfer stresses to adjacent sets. The "dutchmen" sets were horizontally braced with 8-inch, wide-flanged, beam braces placed above the traffic pattern. Horizontal bar spiling was not used in this area to protect the crown of the tunnel.

After remining, two sets were insufficient to reach grade and were remined a second time. On remining, the squeezing ground again moved in. This added pressure was demonstrated by the extrusion of material through windows cut in the liner plate.

The failure of the modified support system was quite sudden and first revealed to the men working at the face by the failure of the power system. The initial collapsed area spread both ways and, within a few days, the collased zone was 120 feet long. Further spreading was contained by the erection of W8X40 A-frames and spreaders inside the standing liner plate sections. Two bucket auger holes, drilled from the surface, provided access for men and materials to the north side of the collapsed zone.

Excavation resumed through the collapsed zone on January 3, 1968 and was completed by May 9, 1968. The support system replacing the deformed sets was four-piece, W10X49, circular ribs installed on 2-foot centers. The sets were connected by 4- by 4-inch, H-beam, collar braces and tie rods. In many cases, W10X49 knee braces also were used. All members of the support system were welded together. As soon as three or four of the circular ribs were in place, the invert was concreted. Then, several heavy coats of gunite were applied to the space between the ribs until a gunite thickness of 10 to 12 inches was obtained. This method was successful in controlling the squeezing ground.

Advance of the heading through the collapsed area was difficult. Initially, the face was breastboarded with a lattice of horizontal 6-, 8-, or 10-inch H-beams and vertical 4- by 4-inch or 6- by 6-inch H-beams. The face was then hand-spaded through the openings in the lattice. Six-inch steel channels were sometimes used to spile ahead of the face. Gunite was unsuccessfully tried as a ground stabilizer at the face.

Ground squeezing became more pronounced as attempts were made to advance through the collapsed zone. Continual trimming of the squeezing ground was required to place the circular ribs. The rate of squeezing ranged from 1 inch up to 2 feet in a single day. The breastboard lattice of steel at the face was twice destroyed and rebuilt. In one instance, the ground moved into the tunnel 15 to 20 feet in a 48-hour period.

The contractor decided to employ a top heading advance supported by two-piece W10X49 ribs resting on wall plates. Small wall-plate drifts were started at springline, but the heavy pressure destroyed the timber sets as fast as they could be placed.

The contractor then decided to first drive lower side drifts outside the final circular zone of rib supports and above invert elevation. The plan was to follow these drifts with smaller drifts at springline elevation. The top heading then would be mined and supported by semicircular ribs resting on the wall plates placed in the upper side drifts.

The side drifts were supported by 6- by 6-inch H-beams and, after an initial advance of 35 feet, the pressure overstressed the beams and the side drifts were backfilled with reinforced concrete. The upper side drifts then were driven as far as the lower drifts and also were backfilled with reinforced concrete. This system of backfilled side drifts provided a reinforced-concrete footing about 11 feet in height and 4 feet in width for anchoring the top heading support system.

The semicircular rib system then was modified by embedding the wall plates in the upper side drift concrete just outside the ribs and connecting the ribs to the wall plates above springline with welded knee braces. This allowed free access to the invert ribs for later connection to the arch ribs at the normal springline location. This procedure was successful and was continued through the remainder of the collapsed zone. Altogether, 69 of the circular rib sections were placed in the zone.

Ground movement during the remining created some surface subsidence, first indicated by surface cracks immediately above the collapsed zone. This surface subsidence progressed, eventually forming a flat inverted cone with a maximum subsidence of 2.8 feet. Accompanying horizontal movements in both directions of less than 1 foot may have resulted from differential settlement.

Following the remining, advance of the south heading was resumed. On October 17, 1969, a 9-inch pilot hole from the south heading intercepted a pilot drill 61 feet in advance of the north heading. The dynamite charge, which enabled the tunnel to be holed through was ignited on October 23, 1969. There was a total of 425,000 cubic yards of tunnel excavation.

Concreting operations were carried on from the north portal with invert concrete being placed from the north portal to the south portal. Arch concrete then was placed starting at the south portal back to the north portal. During the interval when the new shield was being fabricated for the north heading, some invert concrete had been placed. Concreting methods were conventional, that is, concrete was mixed at a portal batch plant, transported by train, and placed in final position by a conveyor system. A total of 141,000 cubic yards of concrete was placed in the tunnel lining.

Prior to arch concreting where known or suspected void areas existed, holes were cut in the liner plate and concrete pumped into the voids. After completion of the tunnel lining, normal contact grouting was accomplished. However, because of the general denseness of the material to be penetrated by the contact grouting, higher pressures up to 80 psi were used and sand was deleted from the grout mix.

Tehachapi Afterbay

The main features of the afterbay facilities (Figures 199, 200, and 201) are: (1) the transition from the tunnel portal section to the control structure; (2) the control structure which discharges into the main canal section approximately one-half mile in length; (3) the bifurcation of this canal section; (4) the West Branch canal section to Oso Pumping Plant; and (5) the Mojave Division section, the Cottonwood control structure, two energy-dissipating chutes and stilling basins. The "main line" California Aqueduct (Mojave Division) and the West Branch within the afterbay area contain an inverted siphon section for drainage control and vehicle crossings. The Mojave Division section also has a flume overchute for the flow from Little Sycamore Creek and a turnout.

Design. Tehachapi Afterbay is a canal section with a design capacity of 5,360 cfs which bifurcates into the canal sections of the Mojave Division and West Branch of the California Aqueduct.

The Mojave Division, with a design capacity of ,388 cfs, is controlled a few hundred feet downstream from the bifurcation at a twin radial-gated structure. The West Branch, with a design capacity of 3,129 cfs, is controlled at Oso Pumping Plant 1½ miles downstream from the bifurcation. The Afterbay was designed for 5-foot fluctuations in water level. All canal sections of Tehachapi Afterbay have a 3- to 5-foot-thick impervious sublining on both the bottom and sides. This sublining protects the concrete lining from the back-pressures of the high water table during the fluctuation of water levels within the Afterbay.

The alluvial soils, within which the Tehachapi Afterbay was excavated, contained sand lenses susceptible to liquefaction during a major earthquake. Six of sixteen in-place relative-density tests were below 70%. However, as these sands were found in small discontinuous beds or lenses, no precautions beyond the sublining and normal requirements for foundation compaction were considered necessary in the design of the canal sections.

The floor slabs of the afterbay control structure were designed on the assumption that uniform foundation pressure will occur. Two 2-inch, galvanized-steel, vent pipes, one on each side of the structure just downstream of the first stoplogs, extend above the floor slab. These vents provide air access to the underside of the flow nappe during operation of the structure when only the stoplogs are used for control.

The existing corrugated-metal pipe drainage system, installed during the initial excavation, was supplemented with additional drains which flow into the canal section below the control structure.

The control structure forms a transition from the 20-foot-diameter south portal of the Carley V. Porter Tunnel to the rectangular gated section. There is a 6-foot rise in invert elevation. Flow through the structure is governed by two radial gates. The gated section forms a transition to the canal cross section of the Afterbay in 62 feet.

The upstream canal section has a bottom width of 50 feet and side slopes of 3:1. The West Branch has a bottom width of 24 feet with 2:1 side slopes. The smaller sized Mojave Division has a bottom width of 10 feet with 2:1 side slopes.

Tejon Ranch, which owns all of the land adjacent to the afterbay facilities, retained the right of access across all drainage structures. Both Oso Siphon on the West Branch and Cottonwood Siphon and the overchute on the Mojave Division contain provisions for access roads for Tejon Ranch.

Oso Siphon has a total length of 421 feet. The upstream and downstream transitions to the twin 16-foot-square barrels are 44 feet - 6 inches in length and each has a 10-foot change in elevation. Stoplogs are provided at the upstream and downstream ends. Overexcavation of 5 feet was required for the structure. Three feet of stone protection overlies 9 inches of drain rock in the stream channel.

On the Mojave Division, the first structure is the Little Sycamore Creek overchute. The overchute is a 7-foot by 27-foot - 6-inch flume with box sections at the intake and discharge ends for road crossings. The intake is stone-protected. The outlet is a concrete slab stilling basin 32 feet in length. The overchute has a center pier support which was overexcavated on a 3:1 slope to 12 feet below the pier and refilled with compacted material for a foundation base.

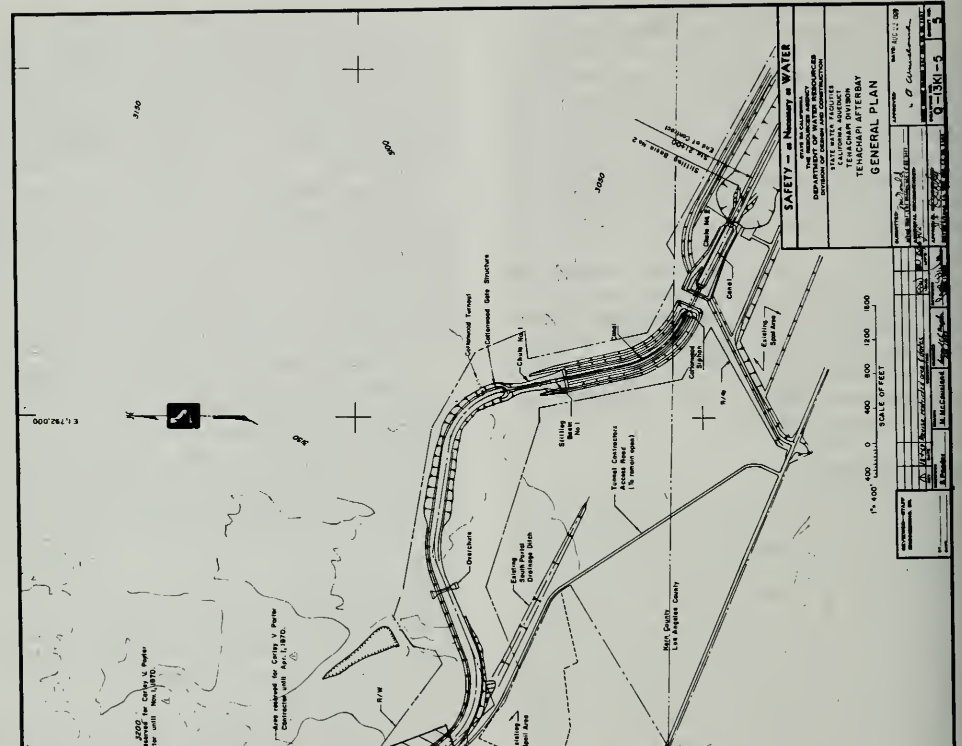

Figure 199. Tehachapi Afterbay—General Plan

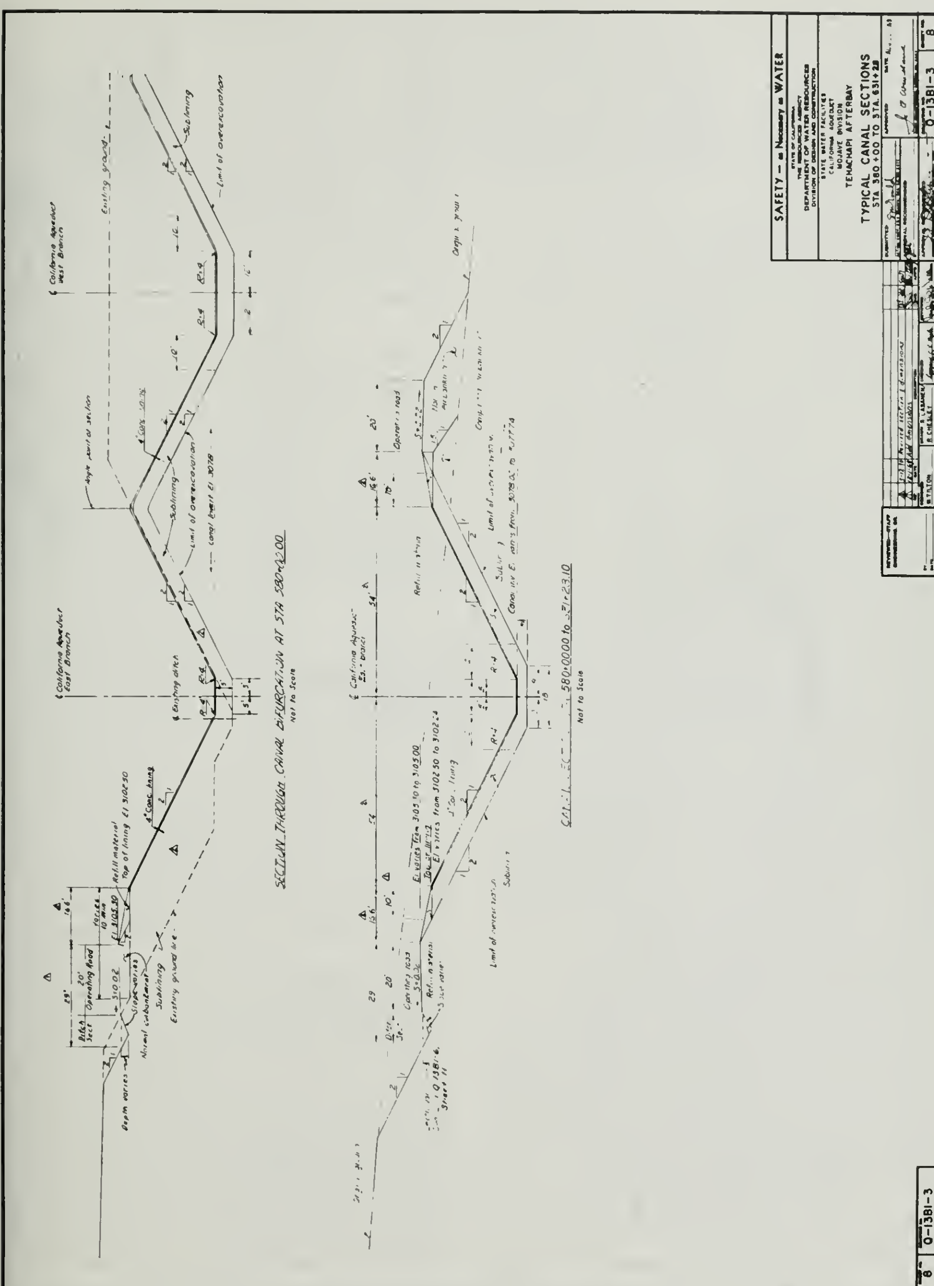

Figure 200. Typical Canal Sections—Tehachapi Afterbay

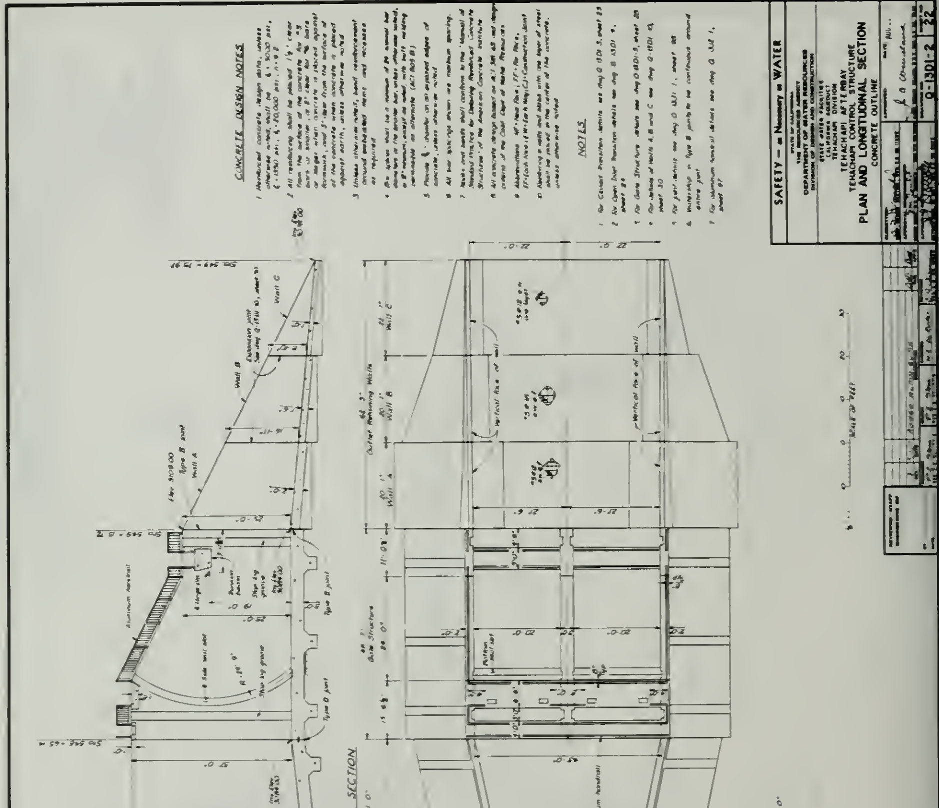

Figure 201. Tehachapi Afterbay Control Structure

The two high-velocity chutes (Figures 202, 203, 204, and 205) were designed to dissipate the energy of 3,500 cfs of water flowing at a velocity of 60 feet per second. At these high velocities, negative pressures will occur at all discontinuous joints. To prevent uplift in the panels or cavitation in the concrete, strict attention to the joint details was required. The concrete finish, particularly of the slab sections, was finished with a minimum of working to avoid a smooth surface. Also, minimum clearances for the reinforcing steel were increased to provide additional protection in all areas where supercritical flows occurred. (See Volume IV of this bulletin for a description of the powerplant facilities that will replace the energy dissipation structures.)

The energy dissipation system starts with an inlet transition section from the canal proper to a twin radial-gated section with upstream stoplogs. The invert of the gate slabs is 8 feet - 9 inches above the invert of the canal. The first chute drops 77 feet - 3 inches to a 120-foot-long stilling basin. Flow is carried in a short canal section to the 240-foot-long Cottonwood Siphon and then to another short canal section and into the final, 86-foot, drop-chute section over a 6-foot weir incorporated in the invert slab. The dentated sill in the stilling basin at the end of the second chute contains two separate sections of baffles for velocity reduction. A short transition section at the end of the basin provides the connection to the regular canal configuration. A valved pipe drain also is incorporated at the head of each chute to completely drain the structure, if necessary. Primary and secondary operating roads, as well as paved drainage ditches, parallel the structure.

The chutes and stilling basins are underlain with an underdrain system to prevent the buildup of hydraulic pressure under the invert slabs. The system was built up from the sublining with a 4-inch layer of filter material, a 3-inch layer of drain rock, and a final 4-inch layer of filter material. Collector pipes underneath the floor slabs and between the shear keys drain the water to longitudinal pipes discharging into the canal. The collector lines contain vent pipes, which also are used as cleanouts.

A 5-foot, reinforced-concrete, pipe turnout is located just upstream of Cottonwood Siphon. The turnout has a standard trashrack and stoplog-protected entrance structure. The outlet pipe is presently stopped with a precast concrete plug for later junction with a water user's connection.

Construction. The canal sections were excavated with conventional tractor-scraper combinations. The excavation did not provide sufficient material for compacted embankments and a borrow area was developed. If the claystone sediments were used for embankment, the concrete lining was required to be placed within 24 hours of trimming to prevent slaking. As an alternative, the embankment after trimming could be sprayed with a liquid asphalt MC-70 or American Cyanamid Arrowspray 52, or equivalent. The contractor used the Arrowspray which worked satisfactorily by providing a tough durable seal.

The proper moisture condition of embankment materials was not achieved in many cases which required that the material be reworked. The poor moisture conditions resulted from the contractor's refusal to use a disc.

A self-propelled multipurpose machine was used for trimming. The machine was rather light but, when the cutting teeth were kept sharp, it performed satisfactorily even where heavy trimming was required. The canal was lined by a belt-fed liner machine followed by finishing and curing jumbos. The liner could not reach the top of the embankment in the wider afterbay sections. In those locations, a finisher was used for final slope paving. This machine also was used for the center invert section after the outer invert sections and slopes had been paved.

Mylar strips, 2 inches in width, were used to form the longitudinal and transverse joints. The longitudinal joint material was fed into the paving machine ahead of the concrete. The transverse joints, however, were placed by a separate machine which traveled up and down the slope between the paver and the final finishing jumbo. This method worked quite well but required considerable hand finishing to obtain a satisfactory joint. The afterbay area is quite windy and, at times, blowing dirt was sufficient to prevent proper concrete finishing. When this occurred, the lining operations had to be suspended. Lining on slope transitions was placed using a slip form paver and a crane with a 1-cubic-yard bucket. The slip form was operated by using two electric winches mounted on the slip form with the cables anchored to the crane.

The foundation slab for the afterbay control structure was placed first. Some difficulty was experienced in placing the 40-foot walls of the open inlet transition. The contractor formed these fairly thin walls, including the counterforts, in a single lift. Consolidation of the concrete was unsatisfactory and considerable concrete repair was required after form stripping.

The invert slabs for the legs of Oso Siphon were placed by a slip form on pipe rails. The contractor had trouble keeping the paver on the rails and there was a tendency for the slip form to sag in the middle. This made screeding difficult and, in conjunction with the slow placement, the concrete set up before proper finishing. Considerable chipping and grinding of the slab surface were required for satisfactory repair. At the Little Sycamore Creek overchute, the inlet and outlet sections and the pier footing and abutments were constructed prior to the canal lining. The flume and pier were completed after the lining.

The Cottonwood gate structure was constructed without incident. In placing the gate seals, an adjustment was necessary to the seal plate in the bottom slab to prevent the gate itself from striking the sill seal. The forms for the Cottonwood chutes and stilling basins were prefabricated with many of the forms interchangeable. Chute No. 2 was placed first. The shear keys were excavated and placed initially, followed by the filter material and the pipe drain system which was, in turn, followed by the remaining structure concrete. After the structure was completed, the piping was flushed to assure it was open and would drain properly.

During a period of intense rainfall, considerable erosion to earthwork occurred throughout the afterbay area. The inclined portion of Chute No. 2 was eroded from 2 to 6 feet under the invert slabs on both east and west sides. The lower inclined slab was undercut to a depth of 12 feet at one location. As Chute No. 1 was not as advanced in construction as Chute No. 2, erosion damage was extensive but not as severe, and the structure remained on a solid footing. Backfill under the slabs and the drainage system was slowly reconstructed without requiring removal of any of the slabs. Considerable debris was deposited in the siphons, but they were structurally undamaged.

Initial Operations

During initial operation of the Tehachapi tunnels and the Afterbay, no unusual difficulties were encountered, nor have there been any problems beyond routine maintenance. Following the February 1971 San Fernando earthquake, the Tehachapi tunnels were drained for the first time and a visual inspection of their condition was performed for the entire length. The tunnels and connecting siphons were in excellent condition. Subsequent inspections have resulted in the same findings.

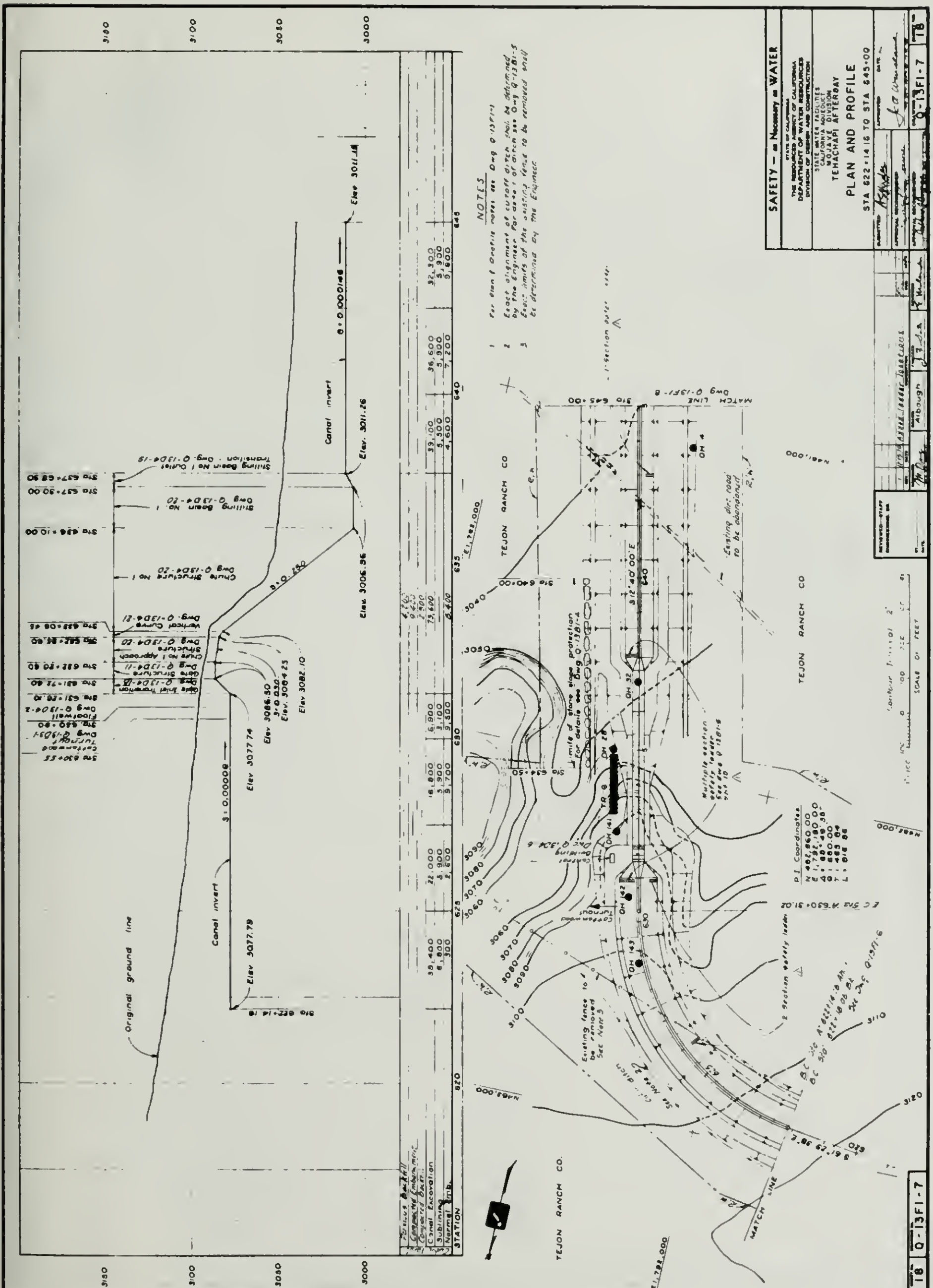

Figure 202. Tehachapi Afterbay—Plan and Profile

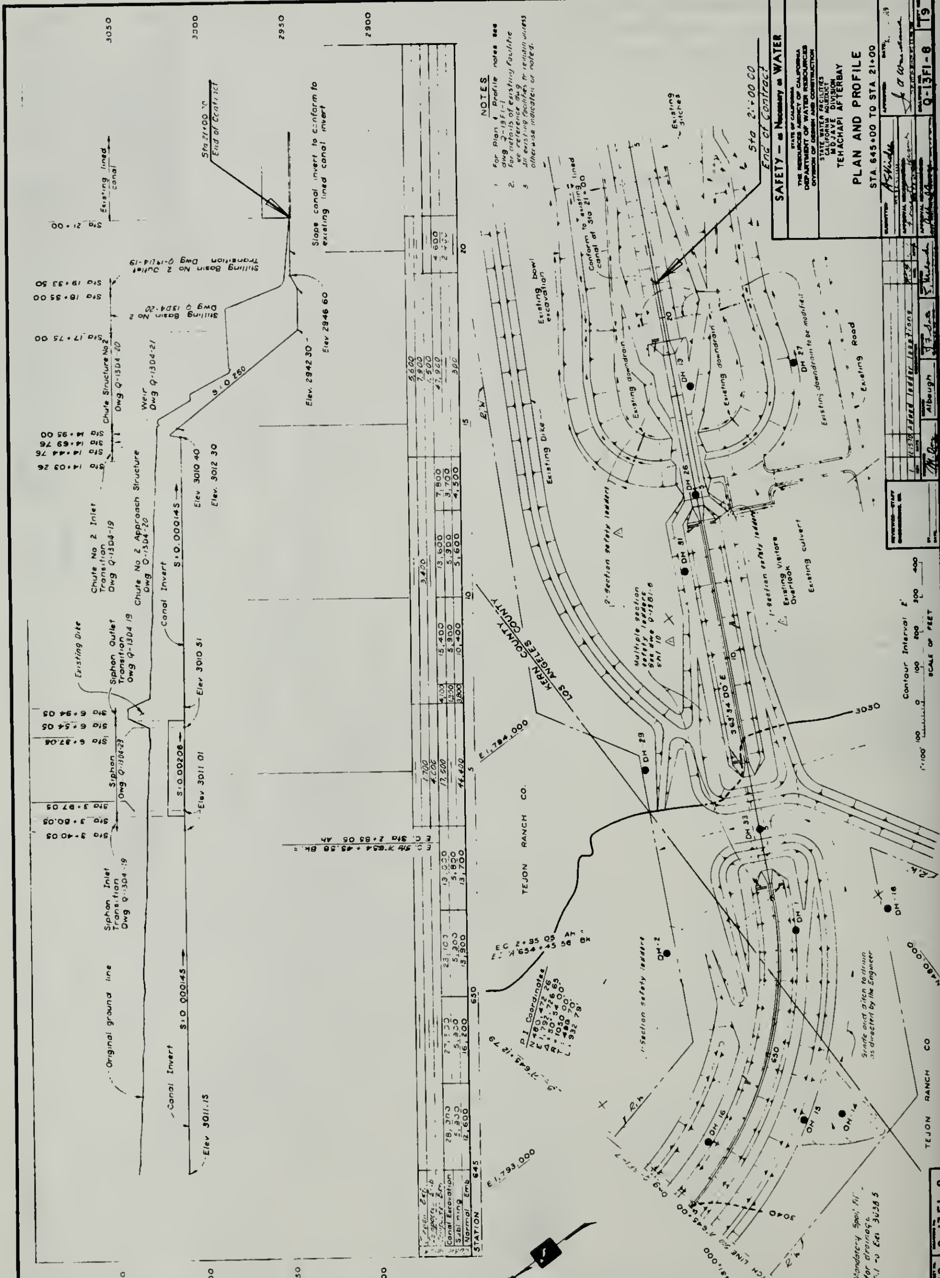

Figure 203. Tehachapi Afterbay—Plan and Profile (Continued)

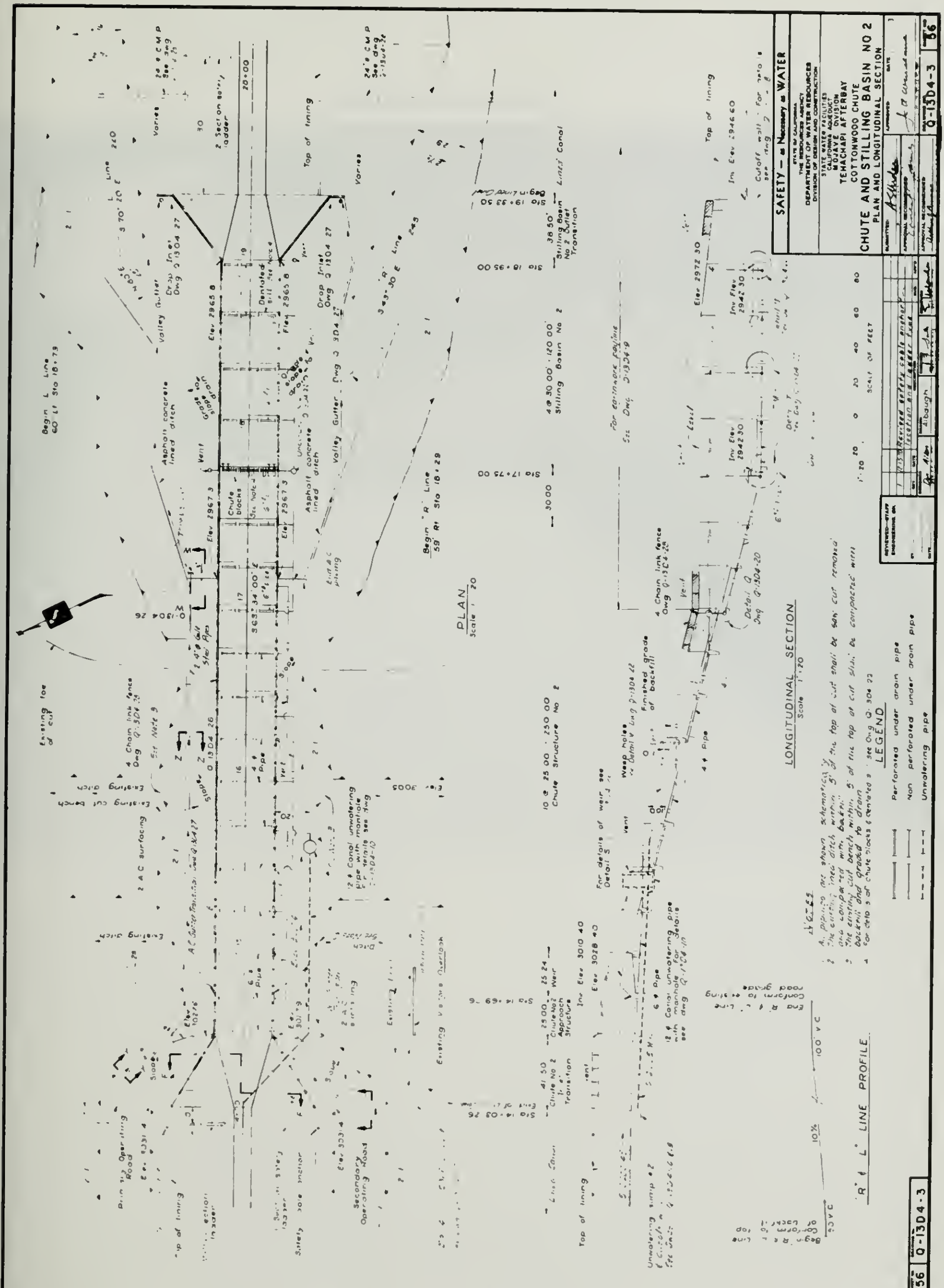

Figure 204. Control Chute and Stilling Basin

Figure 205. Cottonwood Chutes

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Figure 206. Lacatian Map—West Branch

CHAPTER IX. WEST BRANCH DIVISION

Introduction

Role in the State Water Project

The West Branch Division of the California Aqueduct extends 32 miles from Tehachapi Afterbay through Pyramid and Castaic Lakes. Deliveries are made primarily for municipal and industrial water use in Los Angeles, Orange, and Ventura Counties (Figure 206).

Project water flows by gravity from Tehachapi Afterbay through Oso Canal to Oso Pumping Plant where it is pumped into Quail Canal. From there, it flows into Quail Lake, then through an interim outlet facility into a continuation of Quail Canal. Another interim outlet at the end of Quail Canal releases water into Gorman Creek Improvement, a temporary channel which conveys flows into Pyramid Lake until a permanent pipeline and Pyramid Powerplant are constructed.

Water is released from Pyramid Lake through the Angeles Tunnel intake structure into the 37,775-foot-long Angeles Tunnel from where it flows to Castaic Powerplant. Water from Castaic Powerplant is released to Elderberry Forebay, after which it can either be pumped back into Pyramid Lake for pumped-storage operations or released to Castaic Lake where it is delivered to water users.

This chapter describes West Branch conveyance features from the discharge outlet at Oso Pumping Plant to Pyramid Lake. Oso Canal is discussed in Chapter VIII of this volume; Oso Pumping Plant, Peace Valley Pipeline, Pyramid Powerplant, Angeles Tunnel, and Castaic Powerplant are discussed in Volume IV; and Pyramid Dam and Lake, Elderberry Forebay Dam and Forebay, and Castaic Dam and Lake are discussed in Volume III, all of this bulletin. Statistical summaries of West Branch conveyance facilities and Quail Lake storage facility are presented in Tables 19 and 20.

Hydraulic Function

Quail Canal was designed for an ultimate capacity of 3,129 cubic feet per second (cfs). Quail Lake receives off-peak flows from Oso Pumping Plant and, through fluctuation of 11 feet maximum, provides 2,000 acre-feet of usable storage. This allows for continuous deliveries of up to 900 cfs through Gorman Creek Improvement.

Discharge from Quail Lake is regulated by a 108-inch, low-head, butterfly valve in the Quail Lake outlet structure, an interim facility. A short interim reach of canal delivers 900 cfs to Quail Canal.

An interim pipeline from the end of Quail Canal delivers the same maximum 900 cfs through a 78-inch butterfly valve to Gorman Creek Improvement. Gorman Creek Improvement is both a temporary channel to deliver water to Pyramid Lake and a flood-drainage facility.

TABLE 19. Statistical Summary of West Branch

CANAL

Type
Concrete-lined and unlined—trapezoidal—checked

Dimensions
Lined depth, 16.7 feet (upper Quail)—19.7 feet lined and 43.3 feet unlined (lower Quail); bottom width, 24 feet; side slopes, 2:1 (lined) and 3:1 (unlined); length, 2.7 miles (upper Quail) and 2.3 miles (lower Quail)

Capacity
3,129 cubic feet per second

Freeboard
Upper Quail, 2 feet lined and a minimum of 2 feet of earth berm above lining—lower Quail, 3 feet lined and a minimum of 3 feet of earth berm above lining—a minimum of 25 feet in unlined section

Lining
4-inch unreinforced concrete—sealed longitudinal and transverse contraction joints on a maximum of 12½-foot centers

Bridges
2 vehicular

Check Structures
1 three-radial-gate structure (Quail Lake inlet), one 108-inch butterfly valve (Quail Lake inlet), and one 78-inch butterfly valve (at end of Quail embankment) discharging into Gorman Creek Improvement

Cross-Drainage Structures
5 culverts—9 drain inlets

GORMAN CREEK IMPROVEMENT (Temporary):
Shotcrete-lined natural channel; lined depth, 5 feet—bottom width, 8 feet—side slopes, 1½:1; length, 5.9 miles; capacity, 900 cubic feet per second; freeboard, variable; lining, 5 inches of shotcrete with 6-inch by 6-inch, No. 9-gauge, wire fabric

OPERATIONS
Manual on-site control or remote control from area control center at Castaic Operations and Maintenance Center

18—87402

TABLE 20. Statistical Summary of Quail Lake Storage Facility—West Branch

QUAIL LAKE

Type
Compacted embankment dam

Data

Crest elevation	3,320 feet
Crest width	22 feet
Crest length	6,600 feet
Structural height above foundation	45 feet
Freeboard, maximum operating surface	2 feet
Maximum operating storage	5,600 acre-feet
Minimum operating storage	3,300 acre-feet
Maximum operating surface elevation	3,315 feet
Minimum operating surface elevation	3,305 feet

Inlet
Check structure at terminus of upper Quail Canal consisting of three 13-foot-wide by 18.5-foot-high radial gates

Outlet
108-inch butterfly valve to lower Quail Canal

Figure 207. Inlet Structure—Quail Canal to Quail Lake

Figure 208. Quail Lake

Geography, Topography, and Climate

Terrain traversed by the West Branch conveyance facilities is moderately rugged, with elevations ranging from 3,300 feet at Quail Lake to 1,500 feet at Castaic Lake. The area is sparsely populated with virtually no development except for recreational facilities constructed as a result of the Project. Access to these facilities is excellent since State Highway 138, Interstate Highway 5, and old U.S. Highway 99 parallel the aqueduct route.

Temperatures range from about 15 degrees to over 100 degrees Fahrenheit, with hot dry summers and moderate winters. Strong west winds in the canyons, which stem from regional conditions, also tend to carry coastal fogs inland to the Castaic Dam area by way of the Santa Clara River Valley.

Annual rainfall is about 15 inches, concentrated mostly in the wintertime. Heavily concentrated rainfalls have created extensive erosion along Gorman Creek and Santa Clara River Valley. Snow, heavy at times, falls in the higher country with occasional blizzard conditions closing Interstate 5.

Features

Quail Canal to Quail Lake

The West Branch begins at Tehachapi Afterbay and includes Oso Canal and Oso Pumping Plant. From Oso Pumping Plant, water is pumped 231 feet into Quail Canal. This 2.7-mile reach of canal is a concrete-lined channel with a 24-foot-wide invert and 2:1 side slopes, except for cut sections which have 3:1 side slopes. The lining is 4 inches of unreinforced concrete with maximum spacing of transverse and longitudinal grooves at 12.5 feet. The Canal is crossed by a road leading from Highway 138 to a nearby cement plant. The bridge is a reinforced-concrete box-girder structure with five spans.

Quail Lake Inlet Structure

An inlet structure controls flow from Quail Canal into Quail Lake (Figure 207). The first phase of this structure provided for installation of one radial gate. Two additional bays were closed by stoplogs and two remaining gates will be installed in 1975, providing for the 3,129-cfs ultimate capacity. The existing gate is controlled remotely as the remaining two gates will be when they are installed.

Quail Lake

Quail Lake (Figure 208), located on a sag pond along the San Andreas fault, has a surface area of 223 acres and a 3-mile shoreline. A capacity of 5,600 acre-feet was developed by constructing an embankment levee parallel to State Highway 138 on the south.

Quail Lake Outlet Structure

An interim outlet structure (Figure 209) at the southwest end of Quail Lake controls releases into a

550-foot-long interim canal which transitions into a continuation of Quail Canal. Flow is controlled by a 108-inch-diameter butterfly valve in the outlet structure.

A 108-inch prestressed-concrete pipeline carries these flows under Highway 138. The interim outlet and the interim canal will be replaced by a bridge and canal section matching Quail Canal in the ultimate development.

Quail Canal to Gorman Creek Improvement

Quail Canal continues an additional 2.3 miles and is similar in construction and capacity to the reach upstream from Quail Lake. At the end of the Canal, an earth dike diverts the flow laterally into a 78-inch-diameter reinforced-concrete pipeline extending 200 feet to an outlet structure which is the beginning of Gorman Creek Improvement. A 78-inch butterfly valve in the pipeline controls flows to Gorman Creek Improvement.

Gorman Creek Improvement

Ultimate development from the end of Quail Canal to Pyramid Lake includes the Peace Valley Pipeline, which will transport water to the future Pyramid Powerplant. This development will become necessary by 1983 when project flow requirements rise above the 900-cfs capacity of Gorman Creek Improvement.

To conserve project funds during a temporary cash-flow problem, the interim facilities of Gorman Creek Improvement were constructed. They were designed for a short service life, and relatively high maintenance costs were expected. Experience in operation has confirmed this.

The 78-inch valve in the Quail Canal outlet discharges into a trapezoidal drainage channel with an foot-wide invert and $1\frac{1}{2}$:1 side slopes transitioning an 8-foot-wide, $\frac{1}{2}$-mile-long, vertical-wall channel with walls of varying heights (Figure 210). This 000-foot channel also carries runoff from the surrounding areas. Project releases are adjusted to preclude overtopping of the system during storms.

An open channel, protected by riprap, conveys water from a stilling basin at the end of the 8-foot flood channel under the bridge of Interstate 5 to a diversion structure, which turns the flow 90 degrees into the Gorman Creek Improvement channel.

When floodflows from the surrounding areas are larger than 1,000 cfs, excess flows from the diversion structure pass over a concrete overflow levee into the natural streambed of Gorman Creek adjacent to the improved channel.

The concrete-lined Gorman Creek Improvement channel is a trapezoidal section 5 feet deep with an foot bottom width and $1\frac{1}{2}$:1 side slopes (Figure 211).

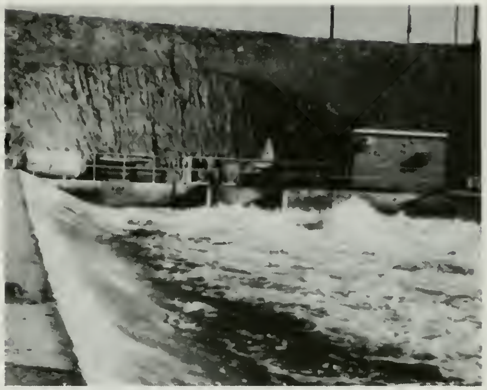


Figure 209. Outlet Structure—Quail Lake to Quail Canal

Figure 210. Gorman Creek Improvement Transition

Figure 211. Gorman Creek Improvement Channel Alignment

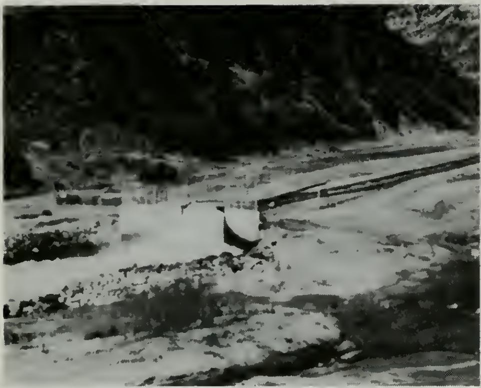


Figure 212. Terminus Energy Dissipotor—Gormon Creek Improvement

Water in this channel flows through several steep culverts and one true inverted siphon at Hungry Valley. It releases into Pyramid Lake through an energy dissipator at the headwaters of the Lake (Figure 212).

Geology and Soils

Geology

The West Branch begins in the western part of the Mojave Desert. Soft alluvial and terrace deposits exist between Tehachapi Afterbay and Oso Pumping Plant, with harder Tertiary sandstones and shales in the low hills traversed by Quail Canal north of Quail Lake. Quail Lake is on the rift zone of the San Andreas fault. The conveyance system leaving Quail Lake extends westward for about 12,000 feet along the rift zone and then turns southward along Gorman Creek, skirting ridges composed of Hungry Valley formation. Gorman Creek flows through Peace Valley, which is underlain by alluvial silty sand and silty valley fill. Approaching Pyramid Lake, sandstones, siltstones, and shales of the Pliocene Ridge Basin group are present. These sedimentary rocks underlie most of the stream channel at the end of Gorman Creek Improvement.

The Tertiary sedimentary rocks south of the San Andreas fault zone are folded rather severely into a series of northwesterly trending folds. This folding dies out in the vicinity of Pyramid Lake, where the rocks have a uniform northerly dip.

Because Quail Lake and some of the conveyance facilities downstream are in the San Andreas rift zone, the possibility of activity along the San Andreas fault system is greater than the hazard of an earthquake emanating from other nearby fault zones, such as the Garlock to the north or the San Gabriel to the west. San Andreas fault movement in this area during the 1857 Fort Tejon earthquake probably had an offset of 20 feet laterally. Since the 1857 earthquake, the seismicity of the area has been low and few seismic events have occurred. However, despite the present low level of seismicity, it is anticipated that a damaging earthquake could occur along the San Andreas fault during the life of the Aqueduct. For this reason, facilities crossing the fault are located on the surface to be readily accessible for repair if damaged.

During excavation of Quail Canal, a number of seeps were encountered in a reach that extended about 3,300 feet along the Canal above Quail Lake. Most of these seeps were along the Tertiary sandstone-terrace deposit contact and appeared to come from perched water trapped in pervious beds within the terrace deposits. An underdrain system was installed throughout this reach to handle the seepage.

Soils

Soils in the alluviated areas of the Mojave Desert mainly consist of silty sand and sandy clays. Excavation through the Tertiary sandstones downstream from Oso Pumping Plant produced earth materials that in large part were suitable for compacted embankment. In one area, a large amount of silty sand was encountered which was placed in a waste area alongside the Canal. All the sandstones were excavated by ripping or common excavation and no blasting was necessary.

The bed of Quail Lake is mostly silts and clays and, therefore, relatively impervious. South of the San Andreas rift zone, excavation in the Hungry Valley formation and in the alluvium along Gorman Creek was in silty sands and silts.

Design

Planning of the West Branch was predicated on ultimate development of its complete power potential, including the existing Castaic Powerplant. As previously mentioned, Pyramid Powerplant and Peace Valley Pipeline will be constructed in the ultimate development.

Quail Canal conveyance facilities were selected on the basis of economic evaluations coupled with other engineering considerations such as topography, geology, location of major faults, and the value of power. Safety against earthquake damage and speed of repair were primary considerations in the selection of the type of conveyance. Below Quail Lake, Quail Canal will have the same water surface elevation as the Lake. A combination earth and unreinforced-concrete-lined canal was found to be the most favorable. This section also provides a regulatory storage capability, which was a necessary factor.

In areas of high ground water, a sand-gravel underdrain system was installed under the canal lining. Dewatering of this underdrain is necessary only when dewatering the Canal. Weekly fluctuations in canal water depth, caused by the power-generation cycle, will be within the impervious earth-lined freeboard and will not require dewatering of the underdrain.

Motor-driven gates are used to isolate Quail Lake from the contiguous canal reaches for maintenance and for emergencies, such as rupture of the Canal from movement along the San Andreas fault. These gates have a rapid closure rate and were designed to close (with manual override) at certain levels of seismic activity.

The invert of Quail Canal from Oso Pumping Plant to Quail Lake is completely in original ground. From Quail Lake through the San Andreas rift zone, the Canal was designed to have the concrete-lined section of the canal prism below original ground. This provides a wide canal embankment and reduces the possibility of embankment rupture throughout the rift zone. Beyond the rift zone, approaching the end of Quail Canal, the entire canal prism was constructed on preconsolidated embankment. This avoids a circuitous on-grade route, thereby greatly decreasing the length of canal.

Gorman Creek Improvement links Quail Canal and Pyramid Lake. The channel siphons were designed for a short service life, originally expected to end by 1978. Project water demands were projected to increase gradually over a seven-year period, starting with 500 cfs in 1971 and approaching the 900 cfs already reached in 1974. The service life now is planned to be extended to 1983 because of adjustments in projected water demands. Accordingly, improvements were made, including increased freeboard and modifications to the outlet transitions from siphons to canal, at the Hungry Valley Siphon, at the energy dissipator into Pyramid Lake at the end of the Canal, and to the fencing.

Construction

Supervision of aqueduct construction in the West Branch was from the Tehachapi-West Branch Project Office, initially located in the San Fernando Valley and subsequently relocated to Castaic. Field offices were located in the Pyramid Lake area at the north portal of Angeles Tunnel and at Gorman, California.

General information about the major contracts for the construction of the facilities for the West Branch Division is shown in Table 21.

Design and Construction by Contract

Quail Embankment

Design. The Quail embankment is about 5 miles southeast of Gorman, 1½ miles south of State Highway 138, and ⅓ of a mile east of Interstate 5 (Figure 213). Comprehensive exploration coupled with conservative design was applied to this embankment to assure that its safety would be at least as good as if excavated in original ground. Drainage facilities near the embankment were extensive and completed in the embankment contract. This embankment is 1¼ miles long and has a maximum height of 45 feet. The embankment was constructed early to allow a 42-week consolidation-settlement period after completing its construction. Canal in the embankment section was trimmed and lined as part of a later Quail Canal construction contract.

Prior to starting construction of the embankment, Southern California Edison Company's 12-kV distribution lines spanning the Canal had to be replaced by a high, long, single span. All other utilities passing through the embankment site remained in place and were protected during construction.

Construction. Access to the work site was from Highway 138 by way of a 60-foot-wide construction access easement. The access easement was fenced and a cattle crossing provided. A permit was obtained from the Division of Highways (now the Department of Transportation) to connect the construction access road to Highway 138. Existing field roads within the Department of Water Resources' right of way were used to reach the contract area, and traffic was maintained across the work site.

The exploration and soils testing program for design indicated that 4.5-foot- to 22.5-foot-deep stripping would be required. The stripped material, with the exception of organic top soil, was moisture-conditioned and placed at 95% of optimum density in the compacted embankment and at unspecified density in the normal embankment. Generally, stripping removed all material having an in-place dry density of less than 100 pounds per cubic foot. To achieve this, in-place density tests were performed for each 1,000 cubic yards of material removed.

TABLE 21. Major Contracts—West Branch

	Specification	Low bid amount	Final contract cost	Total cost—change orders	Starting date	Completion date	Prime contractor
Quail Embankment	66-51	$474,379	$751,464	$157,368	1/17/67	10/18/67	J. E. Robinson Heavy Equipment
Quail Canal West Branch Mile 1.9 to West Branch Mile 8.2	69-28	6,995,635	7,801,762	405,242	1/ 9/70	10/22/71	Altfillisch Construction Co. and Griffith Co.
Gorman Creek Improvement West Branch Mile 8.2 to West Branch Mile 14.1	70-05	2,684,084	2,901,102	72,002	6/ 1/70	3/ 7/72	Griffith Company

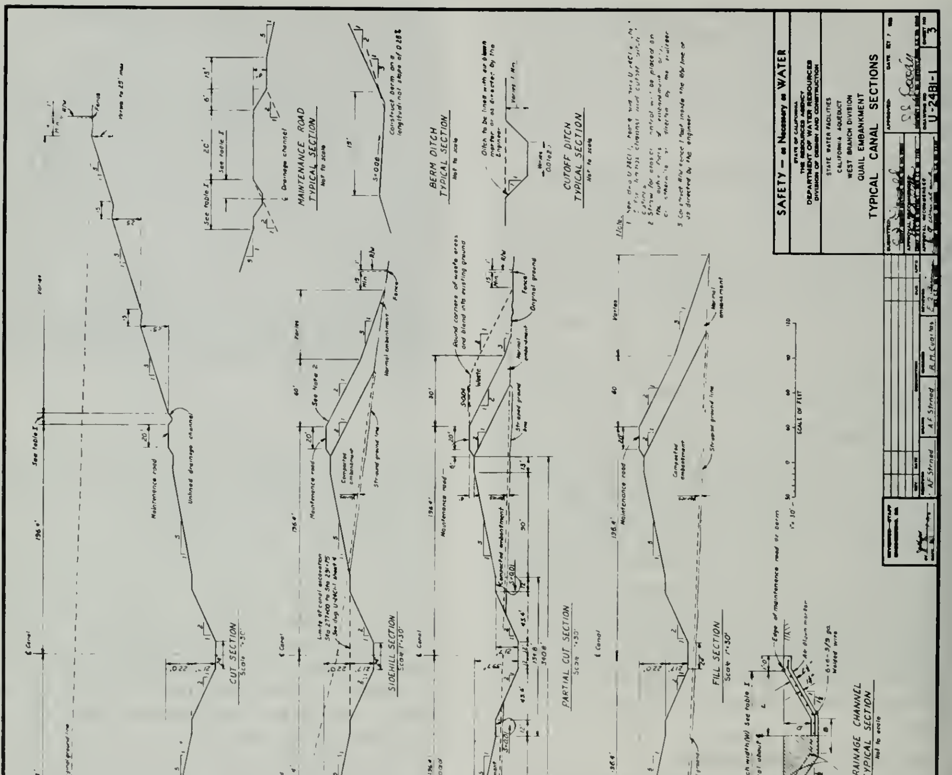


Figure 213. Quail Embankment—Typical Sections

In all, 1,990,500 cubic yards of common excavation was made. Excavated material was used for embankments and fill for drainage areas, and the excess was wasted. The canal prism was excavated in the compacted embankment. A large volume of foundation excavation (stripping) was required because of the low, in-place, soil densities. Excavation was accomplished with the use of 36- and 18-cubic-yard-capacity scrapers assisted by pushdozers.

Quail embankment is composed of compacted and normal fill (Figure 208). About 1,260,000 cubic yards of compacted embankment and 600,000 cubic yards of normal embankment were placed and 40,000 cubic yards of excavated material wasted. A sheepsfoot roller and a self-propelled compactor equipped with sheepsfoot pads were used to compact the embankment.

The compacted fill was near the maximum possible compaction. Materials suitable for this fill were silty sand and sandy silt. Because of the critical nature of the embankment, little latitude was allowed in the selection of possible materials for compacted fill. Except for sandy clay, specific weights of compacted field-dry material were required to be greater than 110 to 115 pounds per cubic foot. Compacted field-dry weight for sandy clay was required to be at least 100 to 105 pounds per cubic foot.

Normal embankment consisted of fills for which no compaction was specified other than that derived from the controlled routing of hauling or spreading equipment. Normal embankment was placed in waste areas and in a large sump area south of Quail embankment so that it would drain. Additional waste areas were provided for disposal of unsuitable, organic, and oversized materials that could not be broken down to the specified size.

Drainage channels, lined with air-blown mortar and reinforced with welded wire fabric, were constructed to provide for diversion of surface stormwater (Figure 214). Cutoff ditches were constructed to prevent erosion of construction slopes. Additional surface erosion control for the embankment was provided by placement of straw.

At selected spots in the foundation, standard Division of Highways settlement-indicating devices were furnished by the Department and installed by the contractor. These devices required careful positioning in the compacted backfill and became an integral part of Quail embankment. They were placed on the foundation of the embankment just after completion of stripping operations.

Quail Canal and Lake

Design. Quail Canal is 6¼ miles long, extending from Oso Pumping Plant outlet works through Quail Lake to Peace Valley near Gorman Creek. Structural features include culverts, drop inlets, drainage ditches, a bridge, a radial gate structure (Figure 207), butterfly valves and vaults (Figure 209), and turnouts. Construction of the Canal required relocation of Highway 138 and Los Robles Road, a private road. The development of Quail Lake as a regulation pool required construction of a large fill in a depressed area to avoid ponding water and a levee on the south side of the Lake.

The Canal and all appurtenant structures were designed to convey 3,129 cfs of project water. Future plans envision even further enlargement of Quail Lake and enlargement of the canal section below Quail Lake to provide water to optimize future power development.

Cut slopes generally were made in 25-foot-high steps with 13-foot-wide setbacks or benches (Figure 215) to avoid excessive excavation. In some areas, the benches were eliminated or widened to handle floodflows. The bench grades are the same as the operating road grades.

In general, overexcavation was required to be carried to a depth necessary to remove foundation soils whose densities were less than 100 pounds per cubic foot. Compacted sublining was required down-aqueduct from Quail Lake. When materials encountered within the canal cut sections, such as clays in foundation overexcavation, met requirements, they were used in the sublining in lieu of specified borrow. Mixing, blending, and scarifying were utilized to meet the specified requirements for sublining material.

Sulfate content of soils in the area ranges up to 50,000 parts per million, and all soil containing more than 1,500 parts per million of sulfate was rejected for aqueduct construction.

A filter-blanketed underdrain system was provided in canal cuts above Quail Lake where high ground water levels were anticipated. Horizontal drains were required in cut slopes where ground water was anticipated above the canal operating road. Fill placed in low areas to permit them to drain was included in

Figure 214. Lining of Drainage Channels

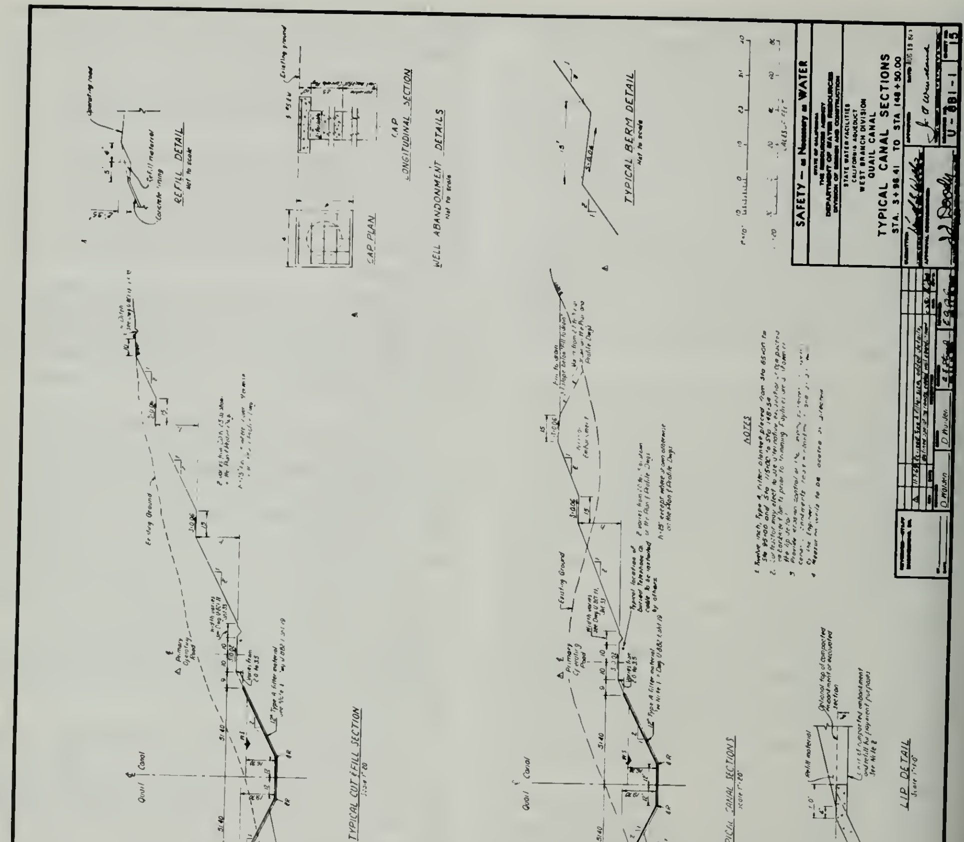


Figure 215. Quail Canal—Typical Sections

normal embankment quantities. Dikes were either normal embankment or compacted embankment, depending on the degree of compaction.

Two types of contraction joints are in the canal concrete lining: sealed contraction joints above Quail Lake and unsealed contraction joints below Quail Lake. Below Quail Lake, the concrete canal lining was a nonsealing wearing surface for the compacted sublining; it was important that there be no joint sealant or waterstop in the joints that would prevent seepage waters from passing through.

The Quail Lake embankment was required to enlarge storage capacity of the original Quail Lake sag pond without flooding Highway 138 and other improvements on the south shore of the Lake. Excavation for the embankment was to a depth sufficient to assure a suitable foundation. The method of determining the depth of excavation was based on the removal of foundation materials that would not consolidate during construction of the embankment. Materials for the compacted levee were obtained from canal, road, and foundation excavations.

A normal embankment on the lakeside toe of the embankment was provided as a stabilizing fill. This was placed to provide at least 5 feet of material above the toe. A large fill placed in low areas on the south side of the embankment drains away from the Lake and conveys floodwaters into the Lake via culverts through Quail Lake embankment. The embankment was cambered to assure proper positioning after settlement, expected to take five years.

Soil-cement protection was placed on the lakeside of the embankment. Insofar as practicable, trees and vegetation were protected and preserved. Certain trees were flagged and specified not to be removed.

Both Highway 138 and Los Robles Road were relocated. A detour was provided for both roads. The Los Robles Road bridge is a cast-in-place box-girder bridge which provides access to the Los Robles cement plant. The Quail Lake operating road provides access around Quail Lake.

Construction. The contract for construction of Quail Canal, Specification No. 69-28, included the construction of a lined canal, inlet and outlet structures for Quail Lake, Quail Lake embankment (Figure 216), Highway 138 relocation, Los Robles Road bridge and Los Robles Road relocation, and other appurtenances.

Canal excavation began in February 1970 and was completed in February 1971. During this time ,300,000 cubic yards were excavated. To accomplish this, the contractor worked a long day shift, 10 and 11 hours, and utilized as many as twenty 40-cubic-yard scrapers. At times, there were three different earthmoving spreads working along various reaches of the Canal. No blasting was required for any of the excavation, but heavy ripping was necessary in the cuts approaching Quail Lake.

Ground water was encountered in the canal prism both above and below Quail Lake. To dewater a 0.2-mile reach above Quail Lake, the contractor installed a system of gravel drains and pipes along the toe of the left side of the Canal. This system permitted completion of the excavation and installation of the permanent underdrain system. The presence of water at other locations gave the contractor no particular problems other than the expected slowing of equipment travel.

Los Robles Road crossing and a gas-line relocation across the Canal below Quail Lake adversely affected progress on the canal excavation. Late relocation of power poles from the canal prism above Quail Lake also resulted in loss of time.

Canal sublining was required in the Canal downstream of Quail Lake. The sublining prism is 10 feet thick for 1.4 miles downstream of the Lake, and the remainder, to the canal terminus, is 3 feet thick. Hauling and placing were done with scrapers and the required 95% compaction was obtained by a sheepsfoot roller.

A filter blanket, 1 foot thick and ⅓ of a mile in length, was constructed upstream from Quail Lake. This blanket lies directly beneath the canal lining and was designed to relieve an area of high ground water. The system includes two rows of perforated pipe under the Canal and three separate sumps for water removal. Filter material for the slopes was placed by a conveyor belt from windrows along the operating roads. Compaction was obtained with a specially constructed vibrating plate mounted on the end of a boom on a piece of grading equipment.

Quail Lake embankment work included excavating 220,000 cubic yards of unsuitable foundation material, placing compacted embankment and normal embankment, constructing six culverts, and surfacing the embankment with soil-cement. Most of the material removed was wasted or placed in normal embankment. The rest was moisture-conditioned for compacted embankment. Other material required for the embankment came from canal and road excavations.

Soil-cement 2 feet thick on the slopes and 1 foot thick on the top was placed on the lakeside face of the embankment. Material for soil-cement came from borrow areas in the Quail Lake basin. Mixing of the soil and cement was done at a central plant located near the west end of the embankment. The feed was continuous, with a twin-shaft pugmill taking material from a soils hopper and cement silo and mixing it with water. A mixture with 12% cement was used initially, but this was later increased to 16% to improve the strength. The material was placed in windrows and spread in 6-inch lifts on the embankment. A spreader box was used with a trailing grid roller for shaping and initial compaction. A pneumatic roller was used to compact the material to the required 95%. Curing was accomplished with sprinklers and a water truck and continued for seven days.

Quail Canal was lined in two reaches by machine;

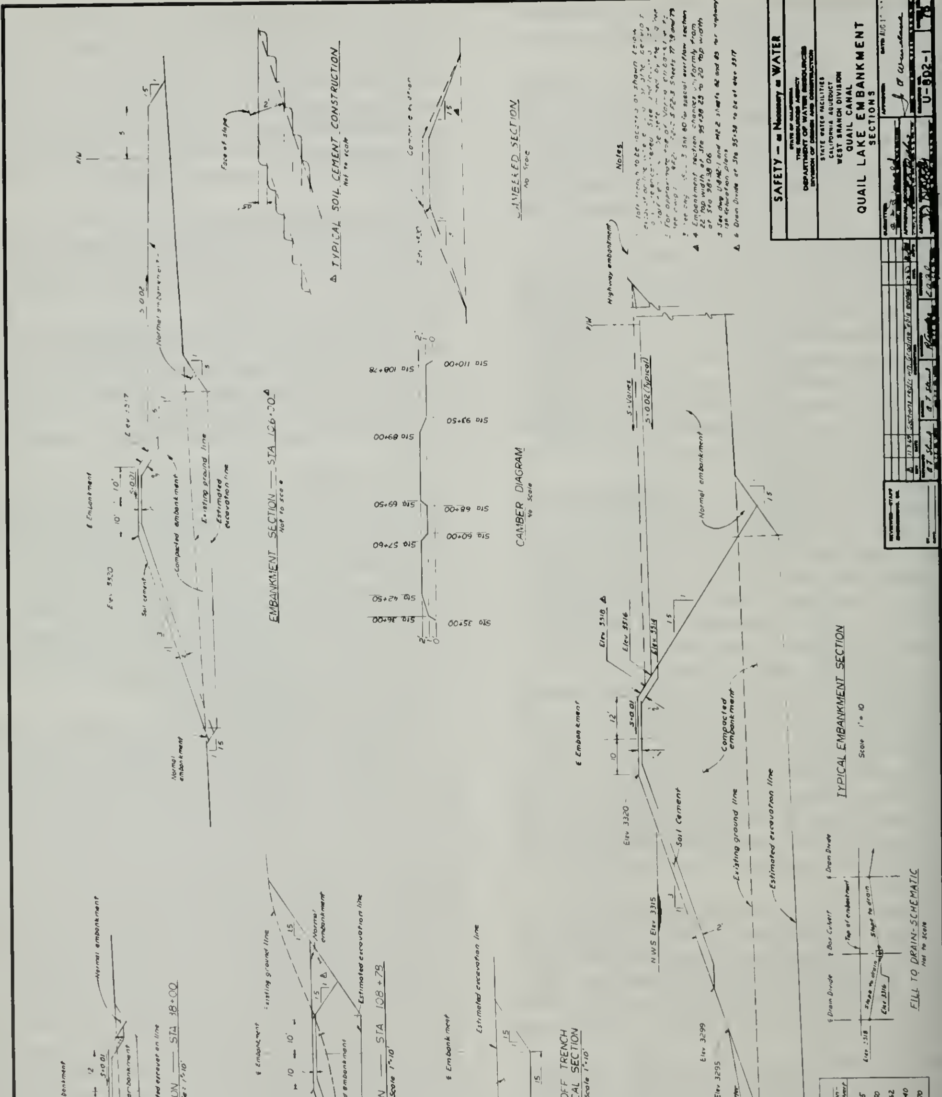


Figure 216. Quail Lake Embankment

the 2-mile reach below the Lake took one month to place, and the 3-mile reach above the Lake took two months. The canal lining is 4 inches thick and required a total of 35,000 cubic yards of concrete (Figure 217).

The paving train consisted of a paving machine followed by a jumbo that inserted the transverse joint material. This jumbo had a sled screed that, as it was pulled up by the slopes, helped shape and consolidate the concrete. The finishing jumbo followed which allowed workers to walk up and down the canal side slopes while finishing the concrete. The last piece of equipment in the train was the curing jumbo for spraying curing compound on the finished concrete.

Two types of joints were used in the canal lining. Joints are unsealed below, and for a short distance above, Quail Lake. A mylar plastic ribbon was inserted by the paving machine at joint intervals to provide a bond break in the lining in these areas. The joints were sealed in the remainder of the Canal.

Hand lining was necessary at the beginning and end of the machine-placed lining and at the structure crossings. A crane was used to pull a vibrating screed up the sides of the Canal as concrete was placed in its path by the crane bucket.

The Highway 138 relocation was necessary for construction of the Quail embankment. The reconstructed road is 6,500 feet long and was designed and constructed to meet requirements of the Division of Highways including future widening of the Highway. The relocation is close to the original highway alignment, and it was necessary to construct a detour around the working area which was in use for 1½ years.

Gorman Creek Improvement

Design. Gorman Creek Improvement extends almost 6 miles from the end of Quail Canal to Pyramid Lake. It consists of turnout facilities from Quail Canal (Figure 218); a reinforced-concrete, open, rectangular, storm channel terminating in an energy dissipator in Gorman Creek; a short reach of improved channel in Gorman Creek; and a reinforced-concrete, multibox, highway culvert-bridge under Interstate 5. At the outlet of the highway culvert, a structure turns the flow at a right angle into a reinforced-concrete-lined, open, trapezoidal channel which includes siphons and culverts at road and stream crossings. The channel ends in an energy dissipator at the upper end of Pyramid Lake (Figure 212). The design conveyance capability of Gorman Creek Improvement was designed for 850 cfs. Under actual operations, with additional freeboard and other minor modifications, 900 cfs is conveyed.

Private access roads are within aqueduct right of way. At these locations, fencing is located between the roads and Aqueduct. This permits the property owners to have access to their remaining property without crossing into hazardous areas.

Barbed-wire fencing originally was installed along the trapezoidal canal to discourage trespassing by people and to bar cattle from the Aqueduct. Later, this fencing was changed to 6-foot-high chain link when the Canal became an attraction that invited trespassing by passing motorists. A 4-foot-high chain-link fence was installed on the walls of the storm channel which carries water at high velocities.

Five channel sections were designed: three open, trapezoidal, concrete-lined sections; one open, trapezoidal, unlined section; and one open, rectangular, reinforced-concrete section (Figure 219). Concrete lining for the trapezoidal channel upstream of Interstate 5 Gorman Creek bridge, where low-velocity flow occurs, is 4 inches thick with expansion and contraction joints. Downstream of the bridge, where flow velocities are over 10 feet per second, the concrete lining of the trapezoidal channel is 5 inches thick without joints and is reinforced with welded wire fabric.

The channel section downstream of Interstate 5 was located so that the top of the lined prism generally is

Figure 217. Quail Canal Lining Operation

Figure 218. Quail Canal Terminal Facilities

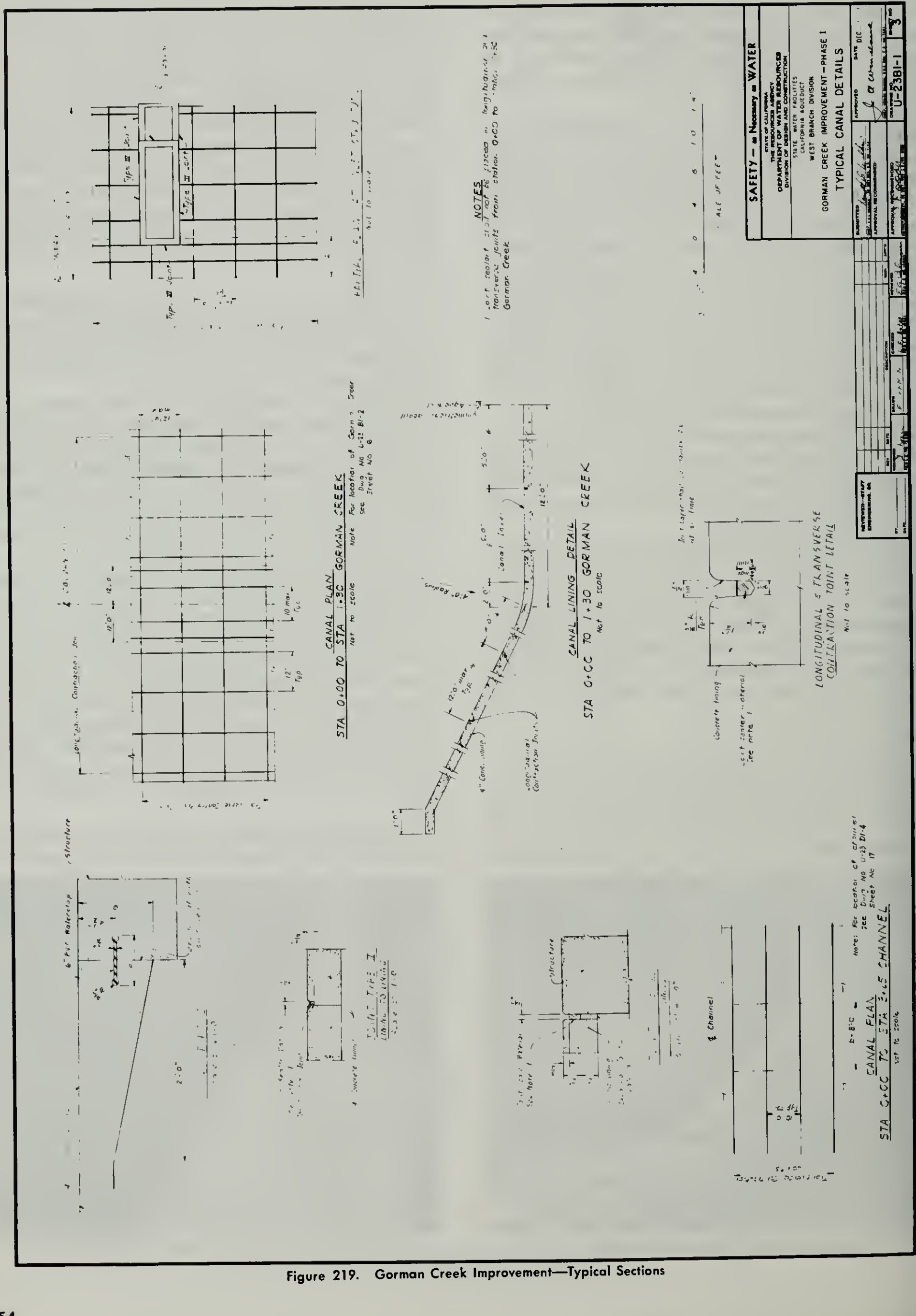


Figure 219. Gorman Creek Improvement—Typical Sections

in original ground. This yielded an appreciable volume of excavated material, which was placed along the channel as normal embankment.

The operating road has a 12-foot minimum width and is 2 feet above the top of canal lining on the normal embankment. Where the channel parallels Interstate 5, cross drainage from the highway culverts is directed into it.

Between Quail Canal and Interstate 5, there are two reaches of 78-inch-inside-diameter reinforced-concrete pipeline (RCP), one leading to, and the other from, the valve vault at the end of Quail Canal.

Below Interstate 5, there are five reaches of 96-inch-inside-diameter reinforced-concrete pipeline across roads and streams. All except Hungry Valley Siphon (Figure 220), the only inverted siphon in the Gorman Creek Improvement, were designed to flow partly full at 850 cfs. Hungry Valley Siphon crosses Cañada de Los Alamos, a deep vertical-walled barranca. Canal fencing and gating were located for access across the pipelines by landowners wishing to cross to their properties; for personnel servicing oil and gas lines and electric transmission lines in the area; and for fire fighting, forestry work, and other public purposes.

Construction. Major components of the work between the end of Quail Canal and Interstate 5 included an inlet structure from Quail Canal that takes the water through a 78-inch RCP; a butterfly valve; another 78-inch RCP to a 6-foot-high by 8-foot-wide, open, rectangular, reinforced-concrete, flood channel 2,300 feet long ending in a stilling basin; 600 feet of riprapped channel; and an energy dissipator near the approach to the Interstate 5 Gorman Creek bridge. This bridge was constructed by the Division of Highways. There also were 8,500 feet of paved access roads constructed in this reach.

Major components of work below Interstate 5 included a diversion structure, 21,000 feet of reinforced-concrete-lined trapezoidal canal, 3,800 feet of 96-inch RCP, and an energy dissipator at Pyramid Lake.

The contractor began clearing and grubbing above Interstate 5. Excavation for the 6-foot by 8-foot rectangular channel essentially was completed one month later, for a total excavated volume of 50,000 cubic yards. The major portion of this material was wasted.

Most of the excavation below Interstate 5 was for the trapezoidal canal, which is 5 feet deep with a bottom width of 8 feet. Total excavation of 300,000 cubic yards required two months. The excavated material was placed as normal embankment along the sides of the Canal for most of the reach.

All open-cut excavation was made with scrapers. No blasting and little ripping was required. Ground water was encountered in the canal prism for a distance of 0.4 of a mile (Figure 221). A backhoe and grader were used to make the excavation in this wet reach.

Three paved roads were constructed to provide access to the canal operating roads and access for landowners south and east of Quail Canal. Two short unpaved sections of road were constructed below Interstate 5 to provide access over the Aqueduct where the original roads were severed by the canal alignment.

Machine lining of the canal prism below Interstate 5 started one week after the trimming machine removed the last 6 inches from the canal grade and placed it in a windrow along the Canal.

Wire-mesh reinforcing was placed in the center of the 5-inch-thick lining after the trimming operation. Laborers with hooks made final adjustments from the front of the lining machine as the concrete was placed. Concrete was produced in a batch plant located south of Interstate 5 and was transported to the site in transit mix trucks. The concrete was discharged onto a conveyor belt which fed the lining machine.

Finishing of the concrete lining was done by workmen from a deck on the rear of the lining machine. Membrane curing of the lining was accomplished by application of a sealing compound from a trailing jumbo.

No joints were made in the canal lining other than the construction joints at the end of a day's run.

Initial Operations

At present, the most advantageous mode of operation between A. D. Edmonston Pumping Plant and Oso Pumping Plant is "off-peak" operation. This utilizes Oso Pumping Plant as much as possible during the off-peak hours of electrical demand, using the storage in Quail Lake and Canal to regulate the intermittent inflow to a steady-state outflow for delivery through Gorman Creek Improvement to Pyramid Lake. This will continue to be the optimum way of operating the West Branch to Pyramid Lake until Peace Valley Pipeline and Pyramid Powerplant are completed. Quail Lake storage will be increased for the ultimate development and will be fully utilized during the buildup to maximum water deliveries.

Initial operations from Oso Pumping Plant to Pyramid Lake uncovered several problems.

Pending installation of two more radial gates in 1975, stoplogs in the two bays of the check structure at the Quail Lake inlet were lowered to allow additional flow through the structure and preclude overtopping of the canal embankment upstream. Prior to lowering the stoplogs, overtopping occurred in 1973 due to this restriction and other causes.

Flows through the Quail Lake outlet structure caused failure of the lining in a short reach of interim canal (Figure 222). Also, a culvert installed at the downstream end of the interim canal blocked flow during high releases resulting in overflow. The outlet at the 108-inch butterfly valve was transitioned with concrete, the concrete spans repaired, and the culvert removed and replaced with a low weir. These modifications corrected deficiencies, and the interim canal

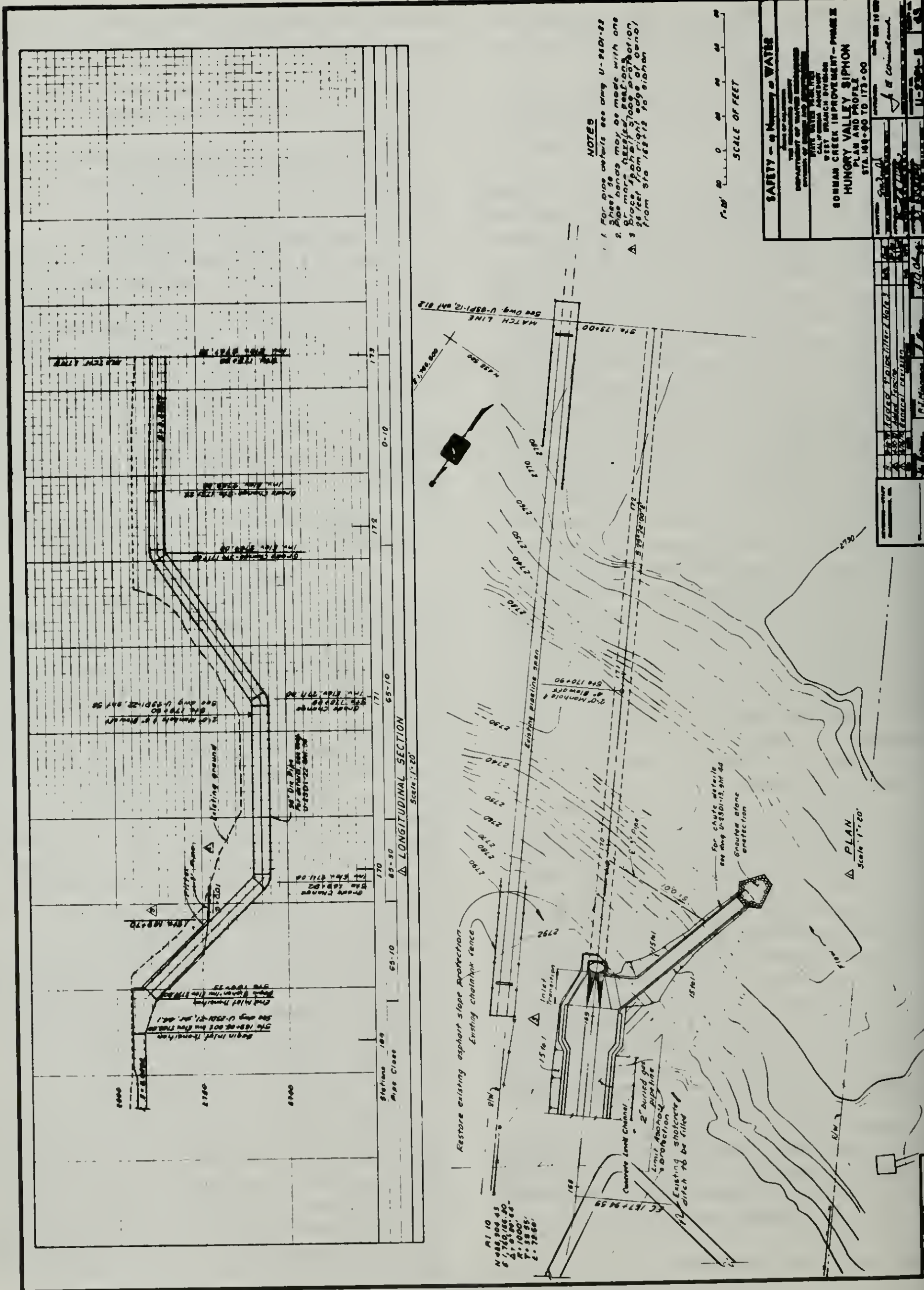


Figure 220. Hungry Valley Siphon

now is adequate to carry the 900-cfs flow on a continuous basis.

Flows from the 78-inch valve at the end of Quail Canal were of such high velocity that they overflowed the channel. The channel walls were raised, which corrected the problem.

Water flowing through culverts and outlets of siphons in the Gorman Creek Improvement resulted in high "rooster tails" and some overflow. Model studies indicated that transitions and additional freeboard would correct this. These changes were installed and overflows no longer occur.

The channel below both energy dissipator structures eroded severely with high flows. This was corrected with installation of riprap and some concreting.

After these and other minor modifications, including repair of leakage in the Hungry Valley Siphon, the entire channel adequately carries 900 cfs and is expected to do so until Pyramid power facilities are in operation in 1982. The relatively high maintenance cost of the temporary system is expected to continue.

Figure 221. High-Water Problem During Construction—Gorman Creek Improvement

Figure 222. Quail Lake Outlet Lining Failure

CHAPTER X. MOJAVE DIVISION

Introduction

Role in the State Water Project

The Mojave Division extends 102 miles from Tehachapi Afterbay on the south slopes of the Tehachapi Mountains to Silverwood Lake, a reservoir impounded by Cedar Springs Dam (discussed in Volume III of this bulletin) on the north side of the San Bernardino Mountains (Figure 223). Water is conveyed along the southern boundary of the Antelope Valley-Mojave Desert with diversions made en route to meet water user needs.

Conveyance of project water through the Division is by canal with inverted pipe siphons used at major valley and stream crossings. At the beginning of the Division, Cottonwood Chutes lower the hydraulic gradeline 140 feet. Pearblossom Pumping Plant, midway in the Division, lifts the water 540 feet (discussed in Volume IV of this bulletin).

Phased Development

The design was based on a three-phase development. Phase I is the present (1974) installation keyed to four pumps at Pearblossom Pumping Plant (two large and two small) for a pumpage of 828 cubic feet per second (cfs). Phase II, to be completed in 1976, is based on meeting projected water demands up to 1990, while Phase III will be undertaken after 1990. Under Phase II, the pumping capacity of Pearblossom Pumping Plant is being increased to 1,380 cfs; additional siphon barrels and check structures are being constructed for this increased discharge.

With regard to Phases I and II, capital investment was minimized in the 1967–1973 period by deferring installation until after 1973 of some features originally scheduled for early construction but not required for initial water deliveries. These deferrals affected the number of siphon barrels and check structures constructed in Phase I as well as the number of pumps at Pearblossom Pumping Plant. Initially, the canal was constructed with sufficient lined capacity to carry Phase II flows. Cottonwood Powerplant will be constructed, when economically justified by the value of the electricity it would produce, to recover the energy loss of Cottonwood Chutes.

Phase III work will further enlarge all conveyance features to carry an additional 700 cfs. The present canal has unlined freeboard to accommodate an increase in the height of concrete lining for this further enlargement, and all crossings over the canal prism are at an elevation sufficient to clear this additional flow. Also, the siphon inlets and outlets have provisions for the additional siphon barrels required for this incremental 700 cfs. However, Pearblossom Pumping Plant will require major modification be-

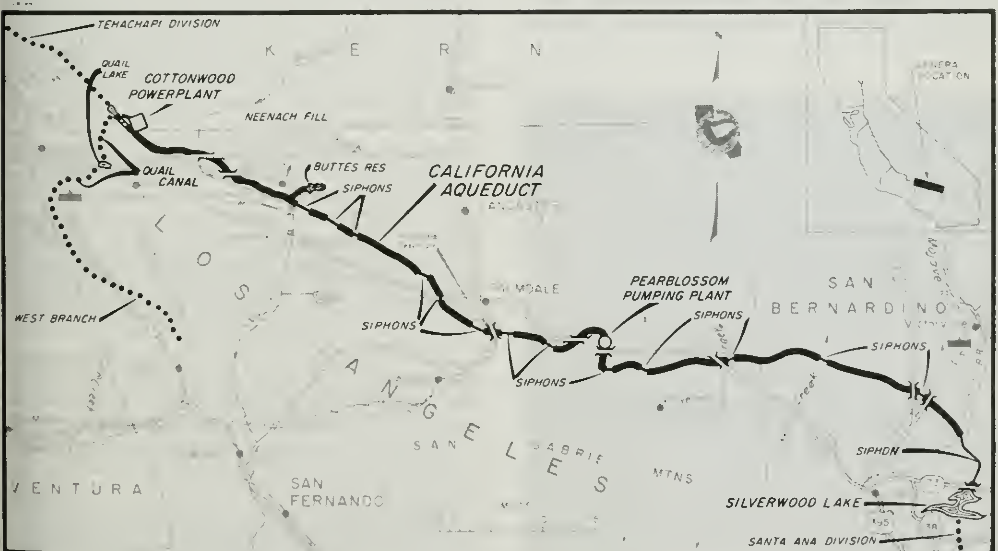


Figure 223. Location Map—Mojave Division

cause the structure does not have space for the additional pumping unit(s) that will be required.

Geography, Topography, and Climate

The Mojave Division is located in Kern, Los Angeles, and San Bernardino Counties and skirts the high Mojave Desert. Although only one or two hours travel time from Los Angeles, the route area is thinly populated. Railroads, highways, and airports provide ready access to the region.

The route topography ranges from valley bottom land with little relief to moderately rugged relief on the north slopes of the Sierra Pelona, San Gabriel, and San Bernardino Mountains. The alignment roughly parallels the San Andreas rift zone and, near Palmdale, lies within the zone for several miles. At route elevation 2,900 to 3,500 feet, Joshua trees are common and, in a few places, exist in dense stands.

An annual temperature range from 20 to 120 degrees Fahrenheit is normal. Average annual rainfall at Palmdale is 13 inches, with rainstorms occurring mainly during the winter. However, intense rainstorms can be expected at any time of the year. Strong westerly winds are frequent and strong north winds commonly occur. The moisture-laden air moved from the ocean by the westerly winds drops most of its precipitation before crossing the mountain barriers to the west of the route area; the north winds are very dry.

Streams rising in the bordering mountains carry drainage across the aqueduct route. These streams enter the valley floor, some in defined watercourses but most in channels that shift from storm to storm. Rainfall often is so intense that the watercourses overflow, covering large areas of the valley floor with sheet flow. These conditions result in changing patterns of erosion and deposition.

Figure 225. Typical Canal—Fairmont to Leona Siphon

Figure 224. Start of Mojave Division

Features

The Mojave Division begins at the bifurcation in Tehachapi Afterbay (Figure 224). The first 36 miles of canal, from Cottonwood Chutes to Leona Siphon (Figure 225), is in trapezoidal canal section with a bottom width of 16 feet and side slopes of 2:1. This reach, from the Tehachapi Mountains to the Palmdale area, approaches and is aligned along Portal Ridge, the south boundary of Antelope Valley.

At Leona Siphon, 4 miles west of Palmdale, the California Aqueduct enters the San Andreas rift zone. The canal follows the rift zone past Palmdale, then leaves it, skirts the community of Littlerock, and continues to Pearblossom Pumping Plant, for a total distance of 18 miles. Side slopes of the canal are flattened to 3:1 and the invert width reduced to 12 feet within the rift zone. The remainder of this reach has 2:1 slopes and a 16-foot invert width.

For 40 miles, from Pearblossom Pumping Plant at the community of Pearblossom to the West Fork of the Mojave River near Hesperia, the canal is a uniform section with 2:1 side slopes and a 16-foot invert width. This reach of canal passes through slopewash from the San Gabriel Mountains and approaches the San Bernardino Mountains. En route, it skirts the Crystalaire residential and recreational development and passes through a thinly populated area southwest of the center of Hesperia. A major siphon, Antelope Siphon, crosses a valley immediately south of Hesperia.

The last 2 miles of the Division is in another major siphon, Mojave Siphon, across Horsethief Creek, a tributary of the West Fork of the Mojave River. This siphon discharges into Silverwood Lake over the left abutment of Cedar Springs Dam. A statistical summary of Mojave Division conveyance facilities is presented in Table 22.

TABLE 22. Statistical Summary of Mojave Division

CANAL

Type
Concrete-lined—trapezoidal—checked

Dimensions
Lined depth, varies from 15.5 to 13.8 feet; bottom width, varies from 12 to 16 feet; side slopes, vary from 2:1 to 3:1; length, 102 miles

Capacity
Variable from 2,388 cubic feet per second below Cottonwood Chutes to 1,225 cubic feet per second through Mojave Siphon

Freeboard
2.0 feet lined and a minimum of 5.5 feet of earth berm above lining

Lining
Unreinforced concrete, 4 inches thick between Cottonwood Chutes and Pearblossom Pumping Plant and 3 inches thick from Pearblossom Pumping Plant to Mojave Siphon—sealed longitudinal and transverse joints on a maximum of 12½-foot centers

Bridges
64 vehicular—1 railroad

Energy Dissipators
Cottonwood Chutes and outlet into Silverwood Lake

Check Structures
20 two-radial-gate structures

Cross-Drainage Structures
102 culverts—86 overchutes—47 drain inlets

Canal Drain
One at Anaverde Creek

Spill Basin
Provisions for a future spill basin at Pearblossom Pumping Plant

SIPHONS

Pipe siphons at Cottonwood Creek, Myrick Canyon, Willow Springs Canyon, Johnson Road, Ritter Canyon, Leona Valley, Soledad Pass, Cheseboro Road, Littlerock Creek, Longview Road (Tejon Siphon), Big Rock Creek, Antelope Wash, Summit Valley (Mojave Siphon), and 3 box siphons in unnamed watercourses between Cottonwood Chutes and Fairmont

OPERATIONS

Manual on-site control or remote control (future) from area control center at Castaic Operations and Maintenance Center

Geology and Soils

Geology

The Aqueduct in the Mojave Division traverses the southern margin of the Mojave Desert near the toe of the San Gabriel Mountains and other mountains bordering the Desert. Much of the canal is on alluvial deposits that cover the desert floor. In the west, these alluvial deposits are mostly silty sands and silts. Farther east, where detritus from the San Gabriel Mountains is washed down on the desert floor, gravels become common although silty sand still predominates. In the vicinity of Fairmont and the Palmdale-Littlerock area, the canal alignment is in foothills and encounters older rocks. The older rocks consist mainly of pre-Cretaceous metamorphic rocks in the Pelona schist and also Mesozoic granitic rocks, mainly quartz monzonite. Some Tertiary sedimentary rocks also are encountered east of Palmdale.

Soils

Mostly, the soils are silty sands but include silts, silt-sand mixtures, and gravels, particularly east of Big Rock Creek.

The deep and shallow subsidence that occurs in the San Joaquin Valley is not found in this reach except for a small area of shallow subsidence at Littlerock.

Seismicity

The San Andreas fault borders the Mojave Desert to the south of the conveyance system and never far from it. The canal crosses the San Andreas fault twice near Palmdale: once in Anaverde Valley and again at Barrel Springs. Other smaller faults, probably associated with the San Andreas system, cross the Aqueduct. Although no major earthquakes have occurred along this portion of the San Andreas fault since the Fort Tejon earthquake in 1857, the fault is considered active, and earthquakes may occur along this part of the San Andreas fault during the life of the conveyance system.

Design

Design of the conveyance facilities generally was guided by the criteria discussed in Chapter I of this volume. However, special conditions required modifications which are described in the following sections.

Hydraulics

Hydraulic design of the Mojave Division conveyance facilities was influenced by two special factors: (1) a decision to provide for a future 700-cfs increase in conveyance capability, and (2) an appreciable reduction in height of Cedar Springs Dam. These became factors late in the final design period.

The 700-cfs increase in capacity without a change in canal slope, invert width, or side slopes required an approximate 3-foot increase in normal water depth. A primary consideration in redesign was to minimize the effect of this increased discharge on the height of pump lift at Pearblossom Pumping Plant. Provisions were made for this change in water depth between Cottonwood Chutes and Pearblossom Pumping Plant by (1) retaining the previously established invert elevations, (2) constructing the larger embankment and excavated sections needed to accommodate the additional flow depth, and (3) redesigning aqueduct overcrossings for additional clearance of 3 feet. Between Pearblossom Pumping Plant and Mojave Siphon, the additional depth was obtained by lowering the invert elevations 3 feet below the previously established elevations.

The maximum normal water surface elevation at Silverwood Lake controlled the hydraulic design upstream to Pearblossom Pumping Plant. Late in 1965, a decision to reduce the height of Cedar Springs Dam about 90 feet required reconsideration of the alignment between Pearblossom Pumping Plant and Silverwood Lake. Studies were made of alternative alignments and pumping schemes. Final design and construction proceeded on the alignment found to be most engineeringly feasible and economical. Essentially, this alignment was the same as the one previously established. The additional head available was used to reduce the pipe diameters of Antelope and Mojave Siphons.

Canal Lining

From Cottonwood Chutes to Pearblossom Pumping Plant, the concrete lining thickness of the canal is 4 inches. Between Pearblossom and Silverwood Lake, the lining was reduced to 3 inches. This reduction in thickness was made in the interest of achieving maximum economy after determining that a 3-inch-thick concrete lining would be adequate in the reach downstream from Pearblossom Pumping Plant.

Canal Check Structures

The spacing of checks was based on operational considerations with the slope of the canal a primary factor. Hydraulic isolation of the San Andreas fault crossing also influenced check spacing. This resulted in Phase I spacing of 2 to 6 miles between checks from Cottonwood Chutes to Pearblossom Pumping Plant and 4 to 17 miles between checks from Pearblossom Pumping Plant to Silverwood Lake. The long spacing of checks between Pearblossom Pumping Plant and Silverwood Lake reflects a decision to eliminate from initial construction all aqueduct items not essential to satisfactory operation during Phase I and the consideration that, in an emergency or for maintenance, this reach of aqueduct can be drained by gravity into Silverwood Lake. As part of Phase II work, two additional checks will be completed in the reach between Pearblossom Pumping Plant and Silverwood Lake in 1975 to decrease the maximum spacing.

Siphons

In the reach from Cottonwood Chutes to Fairmont, drainage is passed under the canal prism by culverts, except at three drainage courses crossed by two-barrel box siphons. Ultimately, after 1990, an additional siphon barrel will be required at each siphon. The transition sections from canal to these siphons include bulkheaded stubs for the future installation of a third barrel. Reinforcing steel was extended from the existing siphon structures to tie to future additions. All siphon barrels may be stoplogged both upstream and downstream but are not otherwise controlled.

All other siphons in the Mojave Division are pipe, and flow through each is controlled by two 13-foot-high radial gates with the exception of the uncontrolled Tejon Siphon, which crosses Longview Road a short distance down-aqueduct from Pearblossom Pumping Plant and also, at present, Antelope Siphon near Hesperia. Radial gates now are being installed at the inlet to Antelope Siphon as one of the two checks being completed as part of Phase II work. Transitions to all pipe siphons provide for future installation of an additional siphon barrel. Except for Big Rock, Antelope, and Mojave Siphons, all of these siphons are presently double-barrel, each 13-foot inside diameter, with provisions for an additional 13-foot-inside-diameter barrel. Big Rock Siphon, which crosses Big Rock Creek near Pearblossom Pumping Plant, presently is a single 19.5-foot-inside-diameter pipeline, and the future barrel will be 15 feet in inside diameter. Antelope Siphon, which crosses Antelope Wash near Hesperia, and Mojave Siphon, which crosses Summit Valley near Cedar Springs Dam, presently are single 11-foot-inside-diameter pipelines, and the future second barrel at each siphon will be 8.5 feet in inside diameter. The second barrel of Antelope Siphon together with check gates are being constructed as part of Phase II work. Twelve- or sixteen-inch blowoffs are used on all siphon barrels.

Operating roads are not carried across streams by bridges parallel to the siphons as was done in the San Joaquin Valley. In the Mojave Division, the operating road crosses valleys at grade near the siphon alignment, except that the operating road is interrupted at some valleys where access to the Aqueduct from the existing public road system is considered adequate.

Sediment Traps

The strong prevailing winds in the Antelope Valley-Mojave Desert area move more wind-blown sediments than in the San Joaquin Valley. For this reason, sediment traps were used more extensively in the Mojave Division than elsewhere in the State Water Project. They are located upstream of the inlet transitions to check structures.

The traps are ungrated troughs 8 feet wide and 4 feet deep across the canal invert and extend up the sloping sides of the lining for ease of cleaning. The side slope trough depth ranges from 0.5 of a foot at the top of the lining to 3.5 feet where the side slopes join the invert.

Drop Inlets

A reach of canal between Interstate Highway 15 and Mojave Siphon skirts Hesperia. A master flood control plan for this area is under study by the County of San Bernardino. Results of this study eventually will provide for disposal of floodflows. However, as an interim measure along a 2-mile reach at Hesperia, these flows are brought into the canal by forty-five 3-foot-diameter metal pipes through the canal uphill dike.

Subdrains

Ground water was encountered at scattered locations between Cottonwood Chutes and Pearblossom Pumping Plant and east of Pearblossom at Big Rock and Mojave Siphons. The limited areal extent and quantity of shallow ground water did not justify the extensive subdrain systems used in the San Joaquin Valley. The faulted rock in cut slopes produced local flows. In these cases, subdrain collector pipes were installed to carry the flow away from the canal prism. The same solution was used where springs were encountered under aqueduct embankments. In other instances, an impervious soil bedding was placed under the canal lining. For a ½-mile reach in the vicinity of Littlerock, and also in the Barrel Springs area near Palmdale, the canal lining joint spacing was reduced to 7.5 feet and the joints left open to relieve uplift from high ground water.

Construction

Headquarters for supervision of construction was located in Palmdale with auxiliary offices in trailers located at the construction sites.

General information about the major contracts for the construction of the facilities is shown in Table 23.

Embankments were constructed in advance of work on these major contracts at two locations where there was a potential for foundation settlement. These are: Neenach embankment midway in the first contract reach and the Anaverde embankments, a series of three fills at the beginning of the third contract reach where the canal first crosses onto the San Andreas rift zone, 4 miles west of Palmdale.

Some major highway and railroad bridges also were constructed across the aqueduct alignment prior to canal construction. These bridges included the Antelope Valley Freeway and Barrel Springs Road bridges near Palmdale, the Southern Pacific Railroad Company's bridge near the Los Angeles-San Bernardino County line, the Barstow Freeway (Interstate 15) and U.S. Highway 395 bridges near Hesperia, and numerous other state and county road bridges.

Many utilities were relocated and reconstructed in advance of aqueduct construction. These included both the first and second barrels of the Los Angeles Owens Valley Aqueduct. The first barrel was reconstructed to be a pipe span over the Mojave Division canal, and the second pipe barrel was encased in concrete to pass under the canal.

TABLE 23. Major Contracts—Mojave Division

	Specification	Low bid amount	Final contract cost	Total cost-change orders	Starting date	Completion date	Prime contractor
Aqueduct—Cottonwood Powerplant to Fairmont Mile 307.7 to Mile 325.8	67-10	$7,516,502	$9,012,107	$891,283	11/14/67	4/29/70	Kirst Construction Company and Clyde W. Wood & Sons, Inc.
Aqueduct—Fairmont to Leona Siphon Mile 325.8 to Mile 344.0	68-21	12,585,236	15,839,469	267,934	9/18/68	1/27/71	Granite Construction Company
Aqueduct—Leona Siphon to Pearblossom Pumping Plant Mile 344.0 to Mile 362.5	68-46	12,071,825	13,523,476	685,154	12/30/68	1/17/72	Kirst Construction Company
Aqueduct—Pearblossom Pumping Plant to Los Angeles-San Bernardino County Line Mile 363.9 To Mile 379.8	69-01	9,746,356	9,824,837	65,182	9/ 2/69	11/19/71	Kirst Construction Company
Aqueduct—Los Angeles-San Bernardino County Line to West Fork Mojave River Mile 379.8 to Mile 405.4	69-17	15,258,014	15,838,982	44,870	10/ 2/69	11/10/71	Granite Construction Company
Mojave Siphon	69-30	2,922,758	2,949,133	10,427	3/18/70	9/28/71	Granite Construction Company

Design and Construction by Contract Reaches

Cottonwood Chutes to Fairmont

Design. This reach of canal is 18.1 miles in length with a lined depth of 15½ feet (Figure 226). The invert slope is 0.00006 or about 0.3 of a foot per mile. Three check structures, each with two radial gates, control flow along this reach. The canal is in both cut and fill with an excess of embankment quantities. This earthwork imbalance resulted from the decision to provide a future incremental capacity of 700 cfs by increasing the unlined freeboard from 2.5 to 5.5 feet.

The City of Los Angeles' original Antelope Valley Siphon, a steel pipeline on the Owens Valley Aqueduct, was reconstructed by the City ahead of the canal construction to span the canal about 6 miles downstream from Cottonwood Chutes. The City's new second Antelope Valley Siphon crosses under the canal at Fairmont. This second pipe siphon was installed and encased in concrete by the City prior to construction of the canal.

Nearly all of the canal excavation was in alluvial silty sand except for one small area of weathered granitic rock near Fairmont.

Floodflows originate in Portal Ridge to the south. These flows are carried over the canal in 17 open, rectangular, reinforced-concrete overchutes and under the canal by 8 reinforced-concrete box or pipe culverts. At three locations, the quantity of sheet flow justified training the runoff and siphoning the Aqueduct under these flows with multibox structures (Figure 227).

Earth dikes uphill of the canal in cut sections train and direct the runoff to suitable crossing sites (Figure 228).

The overchutes range in size from a 5-foot-wide by 6-foot-deep rectangular section to a three-barrel rectangular section of 39-foot clear waterway width and a depth of 7.5 feet. There are six pipe culverts—three 48-inch-inside-diameter and three 66-inch-inside-diameter pipes. There are one three-barrel and one four-barrel box culverts, each barrel 4 feet wide by 5 feet high.

Public roadway bridges crossing over the canal were provided at approximately 4-mile intervals. Los Angeles County constructed three of these bridges prior to construction of the Aqueduct. The Department of Water Resources constructed Neenach Bridge, which is a cast-in-place, prestressed-concrete, box girder designed to carry State Highway 138 over the canal. There are six private access bridges which were designed as precast, prestressed-concrete, I-girder bridges with pile-supported center piers and abutments. Three of these bridges have sufficient roadway width to be incorporated into the county road system.

Two turnouts for the Antelope Valley-East Kern Water Agency, consisting of standard stoplogged and gated inlet structures, were included in the contract. One turnout with a capacity of 5 cfs has 80 feet of 24-inch concrete pipe, and the other with a capacity of 50 cfs has 56 feet of 60-inch pipe. Both pipes are plugged for later use.

Construction. Excavation was by conventional methods (Figure 229) with some ripping required. There were five foundation conditions that required overexcavation. These were: (1) topsoil, (2) soil susceptible to shallow subsidence, (3) low-density silt deposits, (4) soil susceptible to piping, and (5) windblown sand.

Topsoil was overexcavated a minimum of 1.5 feet where embankments were constructed on original ground. Low-density silt deposits occur in a bowl-shaped deposit at a location about one-third of the way through this reach. The lineal extent of this deposit along the canal is 7,700 feet. Tests of this material indicated high pore pressures could build up behind the canal lining. Also, it was likely to liquefy during an earthquake. Therefore, this material was overexcavated 5 feet below invert, and the side slopes were overexcavated 10 feet wide on the top and on a 1½:1 slope to the bottom of the invert overexcavation. The overexcavated material was replaced with compacted backfill.

The Neenach embankment had been constructed two years prior to this work. This compacted embankment, 1,350 feet long and about 27 feet high, was a test embankment to identify possible foundation settlement. A four-barrel box culvert was included in the prior embankment work to carry flow in Neenach wash under the canal.

Trimming and lining of the canal were done by conventional methods and equipment (Figure 230). One-half of the prism was lined at a time. The paving machine placed the longitudinal waterstops, and the transverse waterstops were placed by a separate machine.

Heavy rains during January and February 1969 caused considerable damage both to completed and ongoing work. Twenty-one inches of rain fell, and sufficient uplift pressure built up to buckle some of the completed canal lining. The unlined freeboard of the canal was severely eroded, and debris and sediment were deposited throughout the reach to a maximum depth of 13 feet. Near Fairmont at the end of the contract reach, where Myrick Siphon was to be constructed by the next construction contractor, 150 feet of canal lining was undermined and destroyed and the Myrick Siphon site extensively eroded. The siphon area was excavated to the underlying granitic rock, and the excavated and eroded areas were refilled with compacted material.

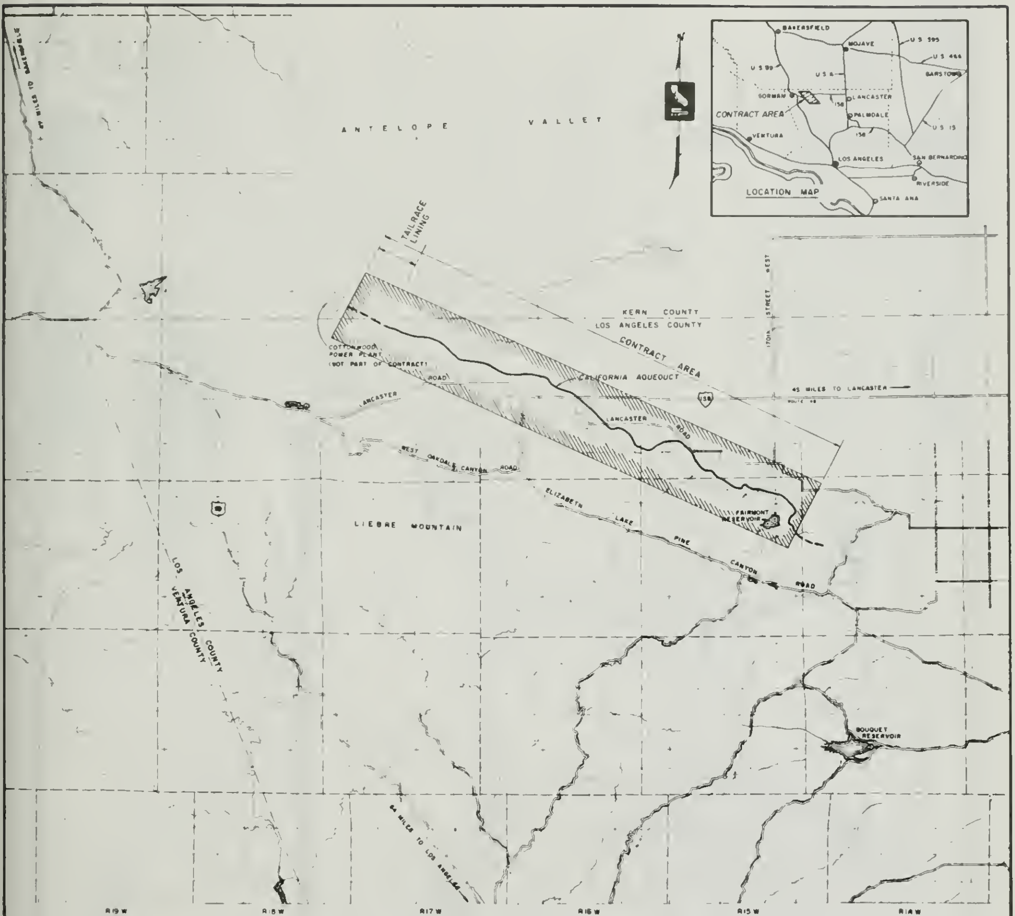


Figure 226. Controct Areo—Cottonwood Powerplant to Leona Siphon

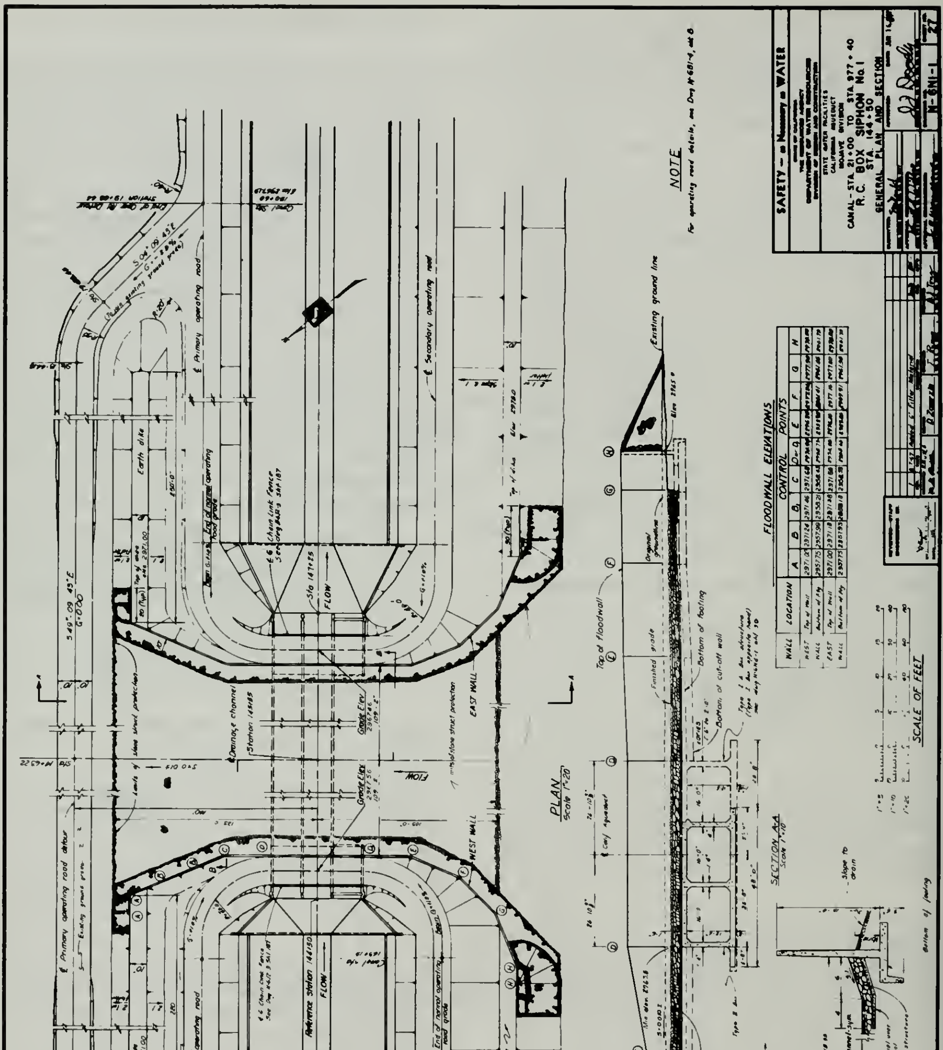


Figure 227. Box Siphon—Generol Plan and Section

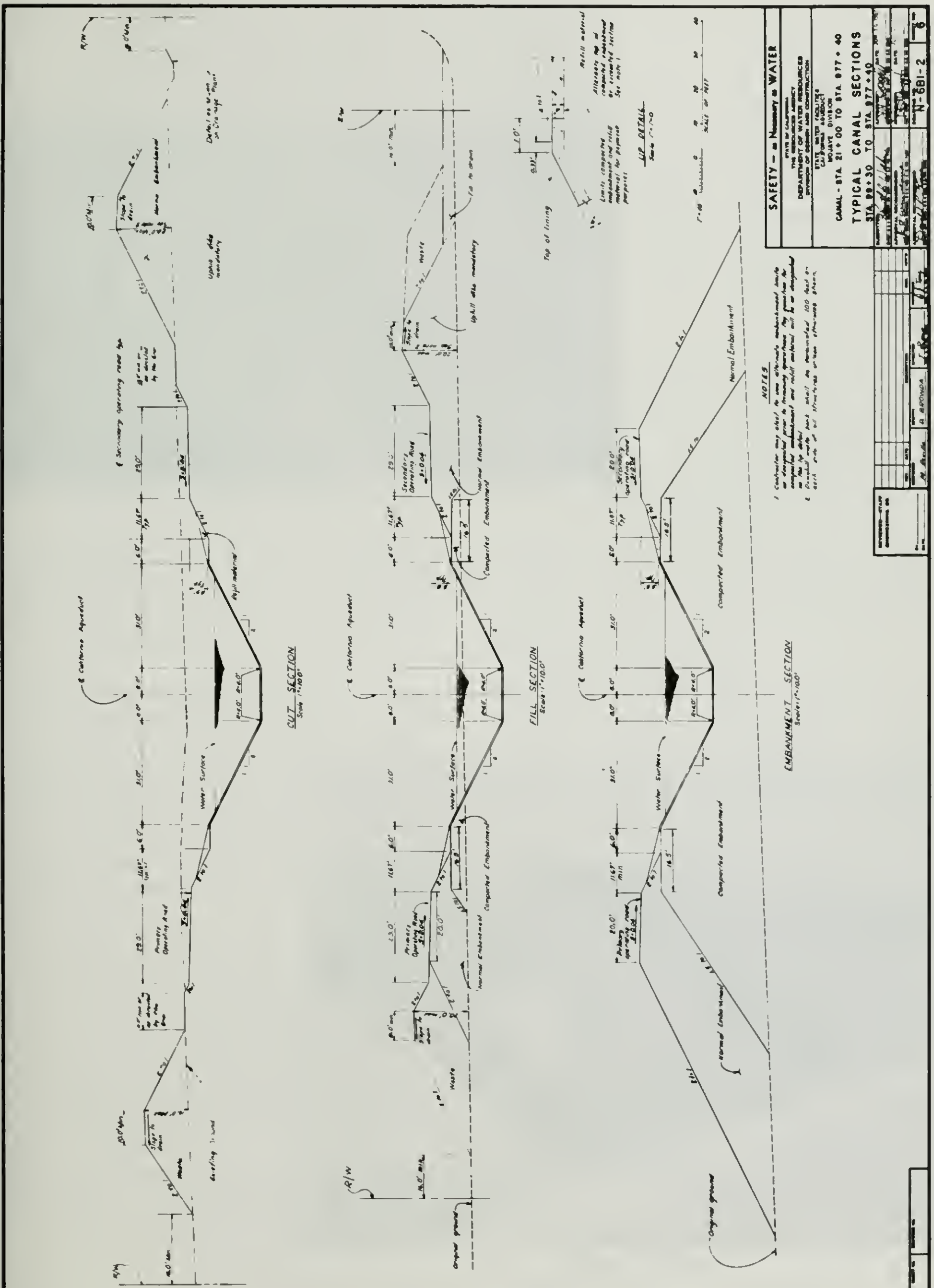


Figure 228. Typicol Conol Sections—Cottonwood Chutes to Fairmont

Figure 229. Canal Excavation

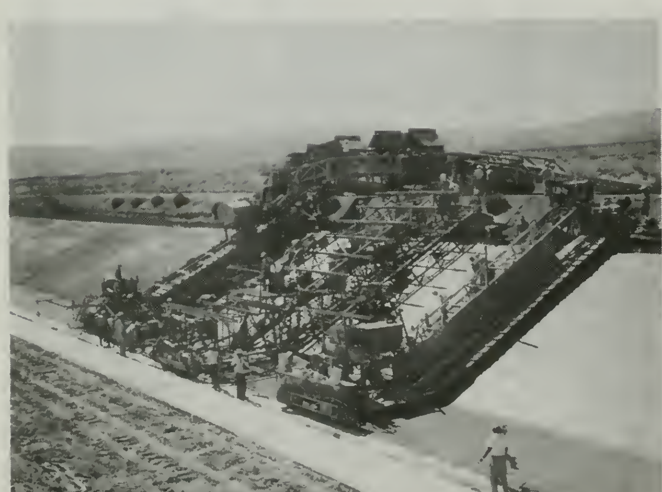


Figure 230. Paving Train

Fairmont to Leona Siphon

Design. This 18.2-mile reach is the second of three contract reaches between Cottonwood Chutes and Pearblossom Pumping Plant (Figure 231). The rugged topography required a sinuous alignment on the slopes of Portal Ridge. Deep cuts and high fills are characteristic of this reach (Figure 232). Except near the community of Quartz Hill, midway in this reach, the route area is thinly populated; land use is limited to stock grazing and, in valley lands, to almond groves.

The reach includes four siphons across major valleys: Myrick Siphon across Myrick Canyon near Fairmont, Willow Springs Siphon across Willow Springs Canyon and Munz Ranch Road, Johnson Siphon across Johnson Canyon and Road, and Ritter Siphon across the Ritter Canyon ecological area 5 miles west of Palmdale. All of these siphons are radial-gate-controlled with upstream and downstream stoplogs. They have two 13-foot-inside-diameter pipe barrels with bulkheaded stubs for a future third barrel.

Tertiary sandstone composed of poorly indurated continental sediments, ranging from coarse conglomerates to fine silts and clays, was encountered in the Willow Springs area. A small percentage of sedimentary rocks consisting of silts, clays, and conglomerates also was encountered. In all, about 85% of the excavation was in rock and the remainder in alluvial deposits, mostly silty sand. Despite the large amount of rock excavated, only a minor amount of blasting was required, and most of the rock excavation was by ripping.

Ground water in rock fractures became springs in some rock excavations, and subdrains were required across the alignment under the canal prism (Figure 233). In special circumstances, a collector subdrain along the canal below invert elevation was required to reach a suitable drainage location. Subdrains also were necessary under some embankments to carry water from seeps across the canal. Subdrains are composed of well-graded sand enveloping well-graded gravel. The widths and lengths of subdrains were determined during construction.

Because of the pronounced topographic relief, culverts crossing under the canal were used instead of overchutes. Fifty-three culverts were needed varying in size and configuration from a single, 36-inch, reinforced-concrete pipe to quadruple, 6-foot-high by 7-foot-wide, box culverts capable of handling 4,500 cfs.

There are seven 20-foot-wide roadway bridges spanning this reach of canal. In addition, prior to construction, the Godde Hill Road relocation and bridge near Quartz Hill were designed and constructed by the County of Los Angeles.

There are two turnouts for the Antelope Valley-East Kern Water Agency. One 85-cfs turnout near Godde Hill Road is a conventional outlet from the canal to a 4-foot-diameter concrete pipe 56 feet long which is timber-bulkheaded for later use. The other is a 10-cfs turnout that taps a barrel of Willow Springs Siphon. This turnout is a 16-inch, mechanically coupled, steel pipe with a 16-inch butterfly valve inside a rectangular, reinforced-concrete, access shaft. This turnout is blind-flanged for later use.

Construction. Between Fairmont and Leona Siphon, nearly 10 million cubic yards of material was excavated and placed in embankment during construction. About 2% of the excavation required blasting of rock ribs which resisted ripping. About 25% of the excavation required multidirectional ripping for removal. Most rocky cuts were stable at 1½:1 slopes and, in alluvium, 2:1 slopes were used. All cut slopes were excavated with 20-foot-wide benches at 40-foot vertical lifts.

Most of the embankments are side-hill fills (Figure 234). The difference in elevation between the surface of the canal operating road and the downhill toe of the fills is as much as 100 feet. These side-hill embankments are notched 6 feet into the original ground to anchor the embankment. Where embankments exceed 30 feet in height, a minimum settlement time of 180 days was specified before lining the canal.

Ground water quality samples were taken at several locations. The water quality was found to be excellent. Therefore, in some locations, weep holes were drilled through the concrete lining or joint material was omitted to permit the water to drain into the canal and equalize the water pressure.

The January and February 1969 rains produced some erosion at the work sites. However, construction was just getting started, and the damage was slight compared to the up-aqueduct reach. Additional springs appeared after these rains, and more extensive use was made of subdrains than anticipated in the design.

In a 600-foot canal reach, about 2,000 feet down-aqueduct of Myrick Siphon, the ground water discharge was between 80 and 200 gallons per minute during the entire construction time. The contractor excavated a wide ditch parallel to the alignment and below invert elevation for dewatering. The foundations for three culverts in this area also required dewatering before construction of the culverts.

The canal trimming and lining equipment was heavy-duty equipment that had been used on the larger canal section in the San Joaquin Valley. The contractor paved the canal in an upstream direction. Trimming and lining methods were conventional. There was no close source of concrete aggregate, and it was trucked 20 miles from the Littlerock Creek area.

Steel H-beam piles were driven to refusal and used to support most of the abutments of roadway bridges over the canal. Three of these bridges had sufficiently good bearing material to use a spread footing for one abutment.

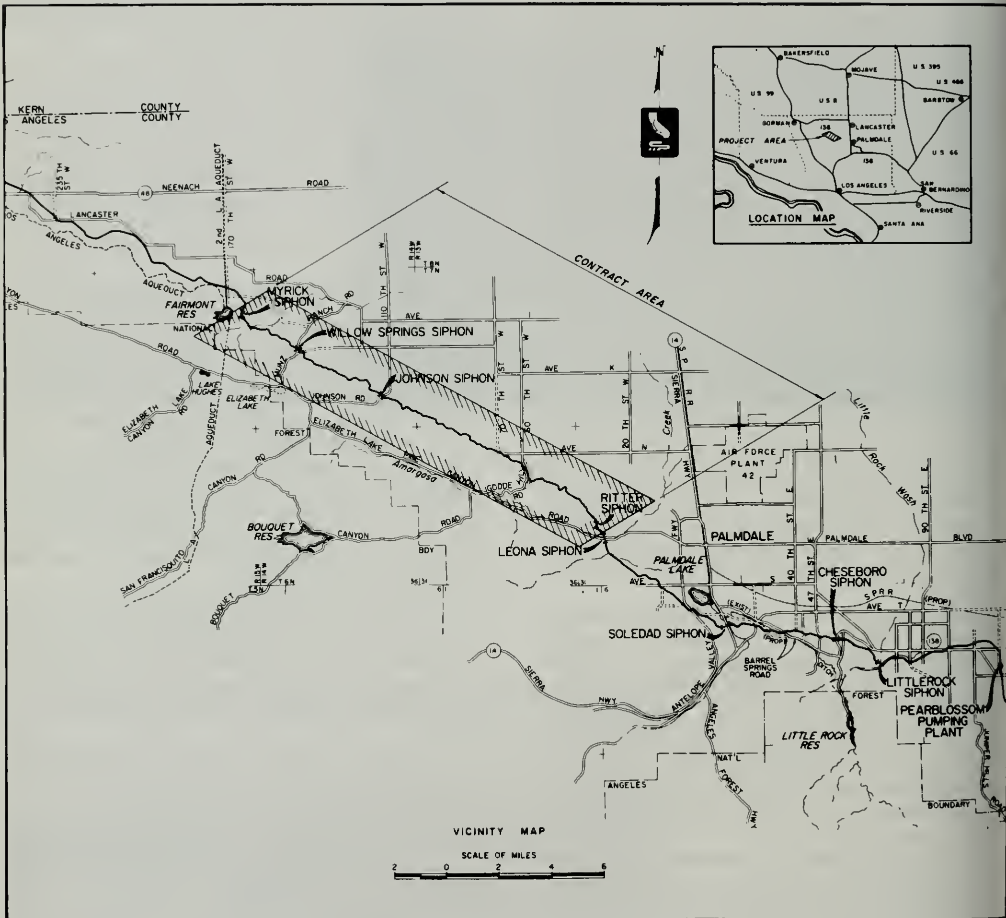


Figure 231. Contract Area—Fairmont to Leona Siphon

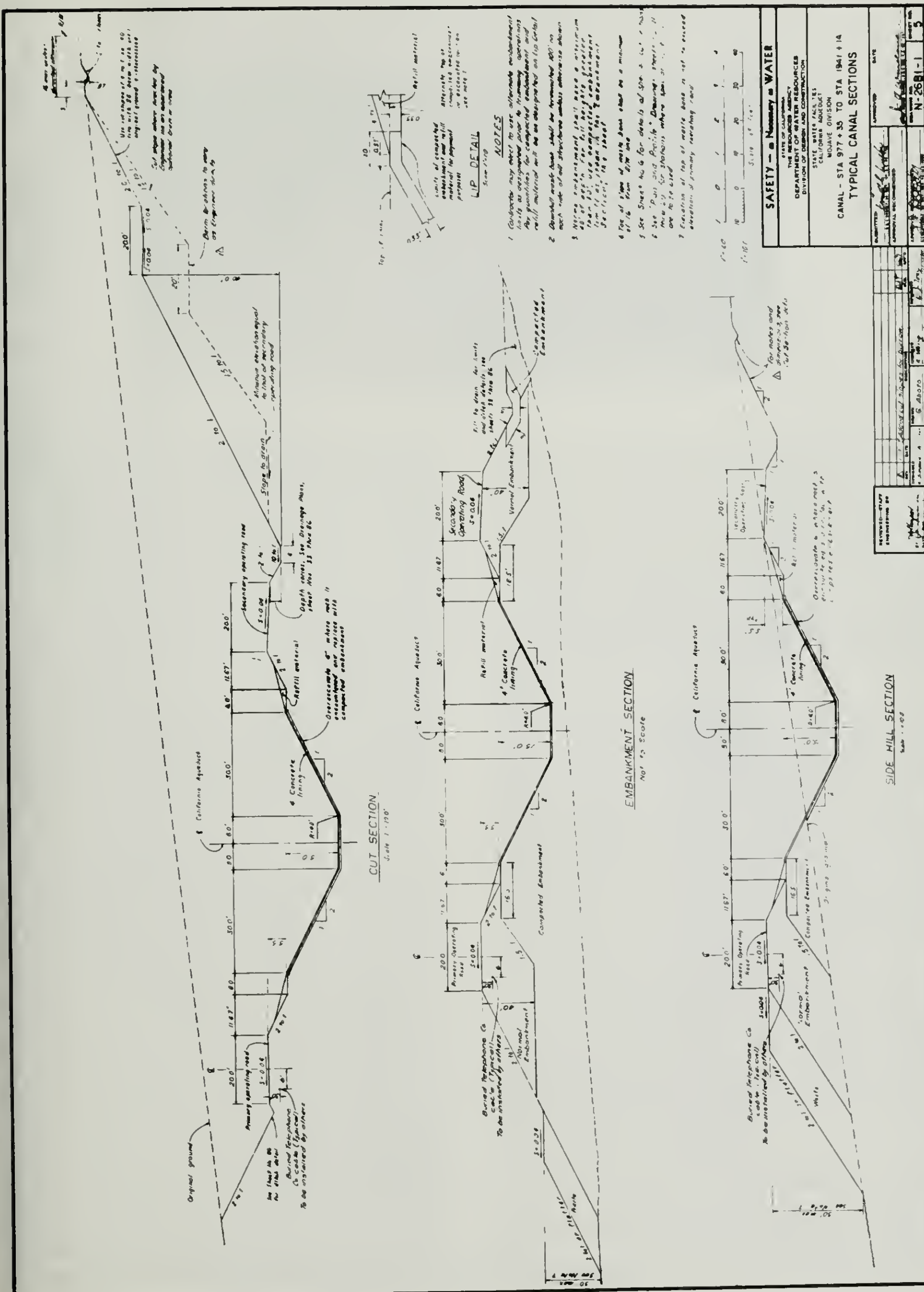


Figure 232. Typical Canal Sections—Fairmant to Leona Siphon

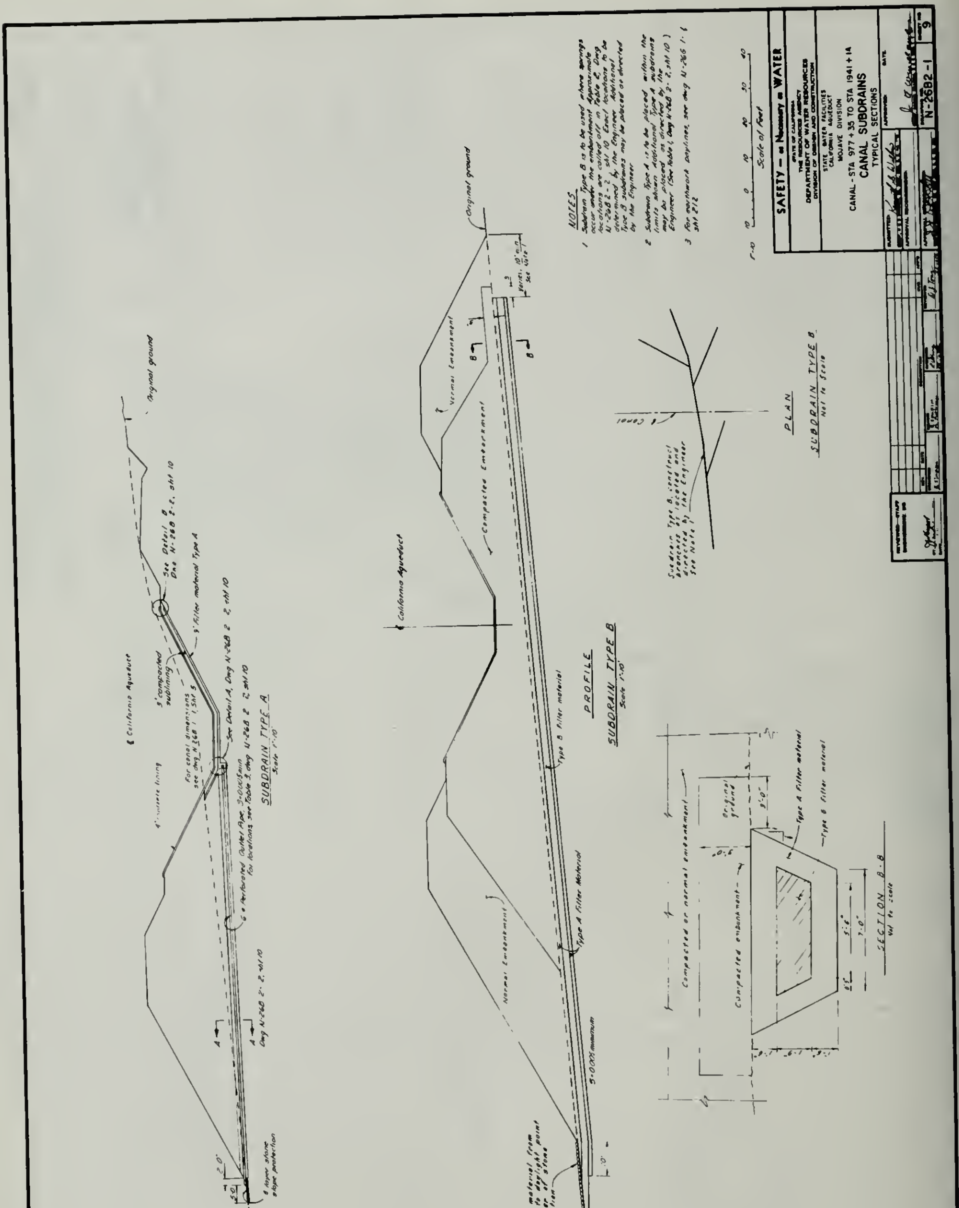


Figure 233. Canal Subdrains—Typical Sections

Figure 234. Canal Alignment

Leona Siphon to Pearblossom Pumping Plant

Design. This 18.5-mile reach of the Mojave Division is the final segment between Cottonwood Chutes and Pearblossom Pumping Plant (Figure 235). This reach differs significantly from the upstream reach: (1) it crosses and follows the San Andreas rift zone; (2) it is largely on the valley floor and therefore drainage control is, in large measure, influenced by sheetflow storm runoff; and (3) it passes through urban areas near the communities of Palmdale, Littlerock, and Pearblossom.

Prior to construction of the canal (Figure 236), there were two preparatory construction contracts. One, near the beginning of the reach, was for construction of the Anaverde embankments, consisting of three separate embankments and drainage facilities at the Anaverde Creek-San Andreas fault crossing. These embankments, which were constructed early to allow time for foundation settlement, were later heavily blanketed on the downhill side with rock, and the canal section in the second embankment was sublined with impervious soil to protect against rupture by earthquake and fault offset (Figure 237). A canal drain also was constructed in the second embankment for rapid canal dewatering into Anaverde Creek in the event the canal is disrupted by earthquake. The other preparatory construction contract was preliminary to the construction of Pearblossom Pumping Plant at the end of this reach and included 2,900 feet of rough canal excavation leading to the Pumping Plant, together with excavation of the pumping plant bowl.

There is a network of roads requiring 18 bridges across the canal—an average of one per mile. This is the closest bridge spacing on the California Aqueduct. Thirteen of the bridges were designed and built by the County of Los Angeles prior to canal construction.

Soledad Pass, near Palmdale, lies between the Sierra Pelona and San Gabriel Mountains and is a major highway and railroad route from Southern California coastal areas. The pass is a short distance south of and above the aqueduct alignment. Highway and railroad requirements, as well as flood runoff from the Pass, posed special design and construction problems for the Aqueduct. Antelope Valley Freeway leaves the Pass and crosses the canal by a bridge constructed by the California Division of Highways (now the Department of Transportation). Sierra Highway and the Southern Pacific Railroad Company's main trackage cross the Aqueduct over Soledad Siphon below the Pass. Runoff from the Pass also is carried over Soledad Siphon.

Three other siphons carry project water across other valleys. Leona Siphon, at the beginning of the contract, is trenched under Amargosa Creek in Leona Valley. A county road also crosses over this siphon. Cheseboro Siphon is trenched across a valley between Soledad Pass and Littlerock Creek and passes under Cheseboro Road. Littlerock Siphon crosses under Littlerock Creek southwest of the community of Littlerock. This creek carries large floodflows from the San Gabriel Mountains into Antelope Valley. All of these siphons are two-barrel 13-foot-inside-diameter pipe with provisions for future third barrels, and all have check gates at their inlets.

Three radial-gated check structures, in addition to those at the siphon inlets, provide for water control and hydraulic isolation of the Aqueduct at the San Andreas fault crossing. Cross drainage, except at siphons, is carried by downdrains, culverts, and overchutes. The overchutes are large-capacity, open, rectangular, reinforced-concrete structures—three, 6 feet high by 10 feet wide and two, 6 feet high by 20 feet wide (Figure 238).

Culverts range in size from single, 48-inch-diameter, reinforced-concrete pipe to quintuple, 7-foot-high by 8-foot-wide, reinforced-concrete boxes (Figure 239). The box culverts have inlets that taper from heights of 6.5, 9, and 11 feet for the 4-, 6-, and 7-foot-high box culverts, respectively. The tapered portions, which have steeper gradients than the main boxes, range in length from 70 to 103 feet. The inlet areas of the culverts and the discharge areas downstream of stilling basins are stone-protected.

The canal drain at Anaverde Creek is down-aqueduct from the trace of the most recent break on the San Andreas fault. This drain is controlled by a slide gate and trashrack mounted over a 3-foot-square opening in the invert of the canal lining. Discharge is into a three-barrel box culvert which carries Anaverde Creek under the canal. In dry weather, this culvert is used as a cattle undercrossing for an adjacent ranch. The slide gate is operated by means of a stem extending to the edge of the operating road.

Initial work was completed on an off-aqueduct spill basin adjacent to the canal up-aqueduct from Pearblossom Pumping Plant. This rectangular basin, which has sufficient capacity to store the equivalent of one-half hour of aqueduct full flow, is intended to capture spill in the event of outage or hydraulic mismatch of the plant. Presently, the basin is rough-excavated only and cannot receive canal spills until a side-channel weir, spillway, and other features are constructed. Completion of this spill basin has been deferred until such time as increased discharge in the Mojave Division and operating considerations demonstrate the need for this spill capability.

Leaving Leona Siphon, the first 9 miles of the reach closely parallels the San Andreas fault and crosses the fault in Anaverde Valley and again at Barrel Springs. Most of the canal is constructed in silty sand but, near Barrel Springs, Miocene shales of the Punchbowl formation and Pliocene sandstones of the Anaverde formation are exposed. Weathered granitic rocks outcrop between Barrel Springs and Littlerock Creek and at Pearblossom Pumping Plant. The remainder of the alignment is in alluvium, mainly silty sand.

There are five turnouts in this reach: one turnout with a capacity of 25.7 cfs is for the Palmdale Irrigation District and Antelope Valley-East Kern Water Agency (AVEK), two turnouts are for AVEK with capacities of 25 cfs each, one turnout for AVEK has a capacity of 15 cfs, and one turnout for Littlerock Irrigation District can deliver 3.5 cfs. Four of the turnouts are standard, slide-gated, gravity turnouts from the canal. The fifth is a 12-inch steel-pipe turnout which taps a 16-inch blowoff on Soledad Siphon; a reducing tee connection, butterfly valve, and turnout pipe extend this fifth turnout to a stone-protected ditch.

Littlerock ditch, which carries water from Littlerock Reservoir on Littlerock Creek to Palmdale, was crossed by the canal and relocated where necessary.

Construction. Test stations installed during fill construction showed minimal settlement in the three Anaverde embankments prior to canal construction. The first embankment crosses the most recent trace of the San Andreas fault. The second and largest embankment, which has a clay sublining under the canal concrete lining, crosses Anaverde Creek and the third crosses a tributary creek.

The canal side slopes are 3:1 from Leona Siphon to Cheseboro Siphon. This reach generally is within the San Andreas rift zone. Beyond Cheseboro Siphon, the canal side slopes are steepened to 2:1. Compacted earth sublining was used where ground water uplift pressure was expected.

Excavation was accomplished without blasting and only ripping was required. Usually, overexcavation was required where rocky foundation or unsuitable alluvium was encountered. Overexcavation ranged from 1 to 4.5 feet below invert elevation and 6 feet beyond the canal sides.

Ground water in the Barrel Springs area near Palmdale required excavation 10 feet below invert elevation and a deep side slope overexcavation for placing an impervious earth sublining. Extensive pumping was required during construction, and dewatering wells 50 to 60 feet deep were placed along the uphill operating road and pumped with portable submersible pumps.

At Leona Siphon, another extensive well point system was required in Amargosa Creek. Well points on 4-foot centers on each side of the pipe trench excavation extended 7 feet below pipe subgrade.

A shoofly was constructed at Soledad Siphon for the Southern Pacific Company's railroad. After completion of the Siphon, the main-line trackage was replaced.

The trimming and concrete lining operations generally were the same as on the previous reaches. A polyvinyl chloride waterstop was used for the longitudinal contraction joints. Where the waterstop was omitted in high ground water areas, a "V"-shaped polyvinyl strip was inserted during paving to form the groove and later removed. The groove then was filled with a polysulfide sealant.

The transverse joints were formed differently from previous contracts. During paving, a vibrating guillotine gouged a 2-inch-deep groove, pushing aside the large aggregate and partially filling the groove with mortar. Additional mortar then was applied to fill the groove and the concrete finished. The joints then were sawed to form a ⅜-inch-wide by ⅝-inch-deep groove and filled with a cold-applied, two-component, polysulfide polymer within 30 hours after the joint was cut (Figure 240). Where the saw cut crossed longitudinal joints, the cut was deepened to sever the longitudinal waterstop, thus providing a sealed contact area between the two contraction joints. Random cracking also was repaired by sawing or by chipping with an air chisel and then sealed.

Because of the numerous bridge crossings, nearly a mile of canal trimming and paving could not be performed by conventional machine methods. The contractor modified a paver to work on slopes. A cutting bar was attached for trimming, and a working platform was attached behind the paver for the concrete finishing workmen.

There are two farm access bridges in this reach which are concrete, cast-in-place, girder type. The abutments are supported on cast-in-drilled-hole concrete piles. After the bridge girders were formed, the deck was placed and struck off with a finishing machine.

Littlerock Bridge on State Highway 138 is supported on steel shells filled with concrete. Spiral corrugated shells are closed at the lower end and taper from 12 inches at the tip to 16 inches at abutment elevation. The bridge structure is cast-in-place, prestressed, post-tensioned, box-girder type.

The Pearblossom Pumping Plant access bridge over the canal has three precast prestressed girders. Cast-in-drilled-hole concrete piles support the abutments.

Because of nearby San Andreas fault, an elastic, pipe-joint, filler material was specified for all siphons in this reach. Several types of sealing materials were tried for hot and cold application. A cold-applied material proved the most satisfactory.

When testing the siphons for leakage, all were satisfactory except Leona Siphon, which was reexcavated, the outside and inside joints refilled with joint filler, and less backfill replaced to reduce pipe deformation. The Siphon then tested satisfactorily.

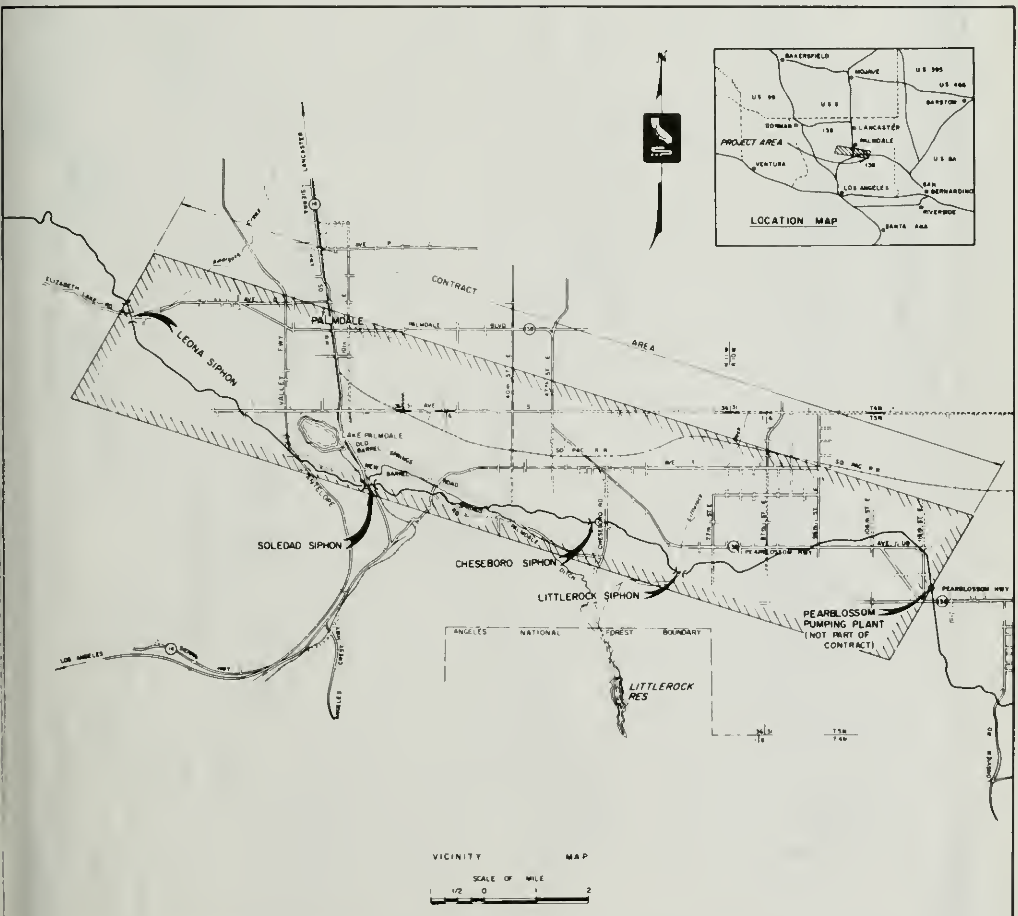


Figure 235. Contract Area—Leona Siphon to Pearblossom Pumping Plant

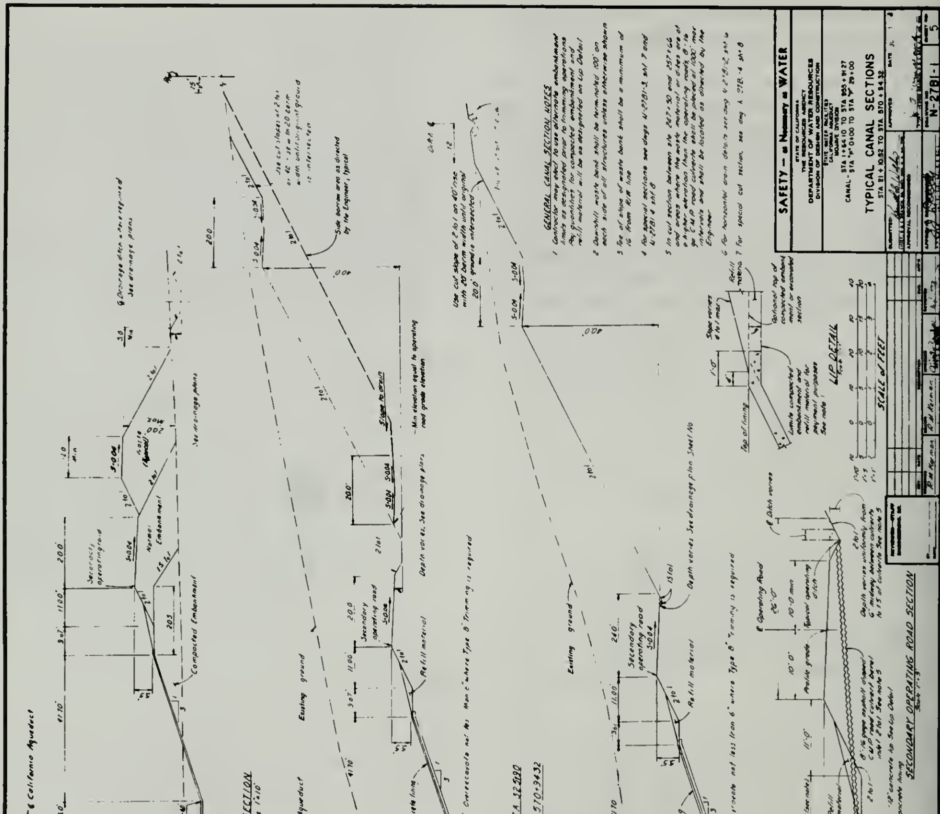


Figure 236. Typical Canal Sections—Leona Siphon to Pearblossom Pumping Plant

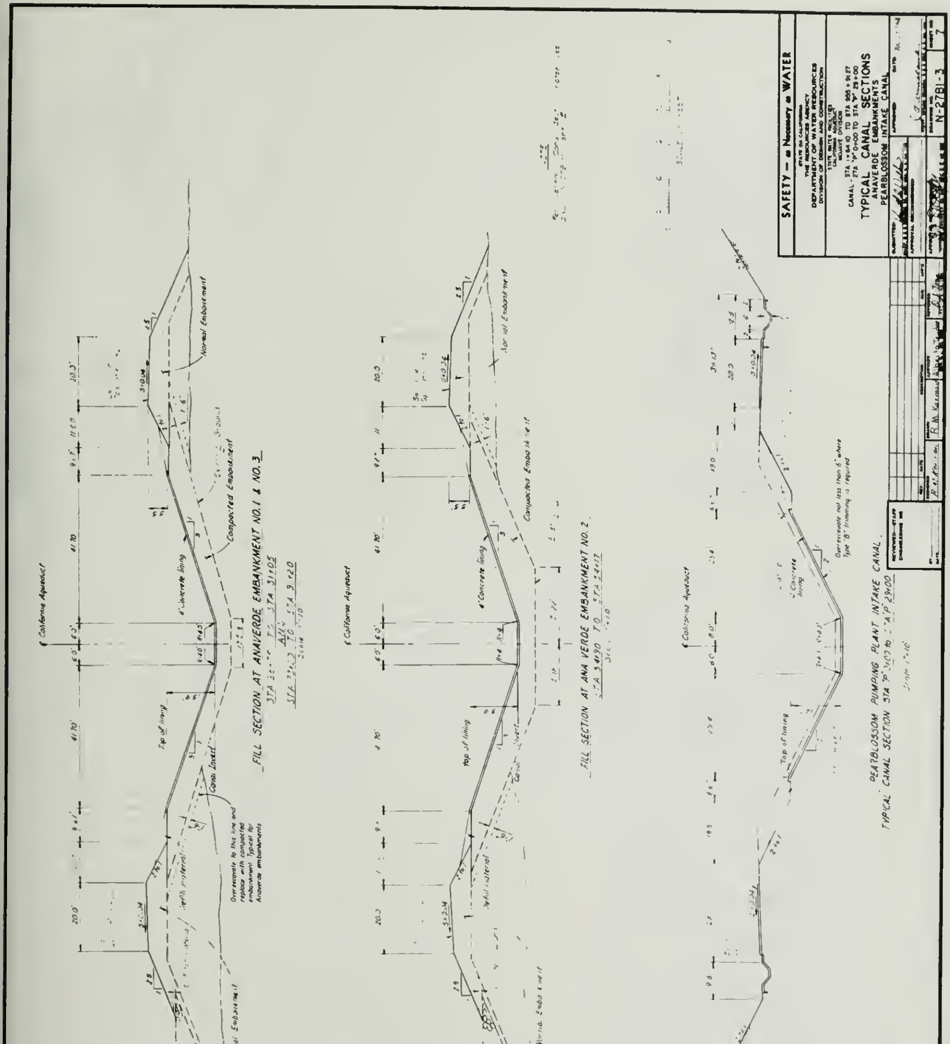


Figure 237. Typical Canal Sections—Anaverde Embankments

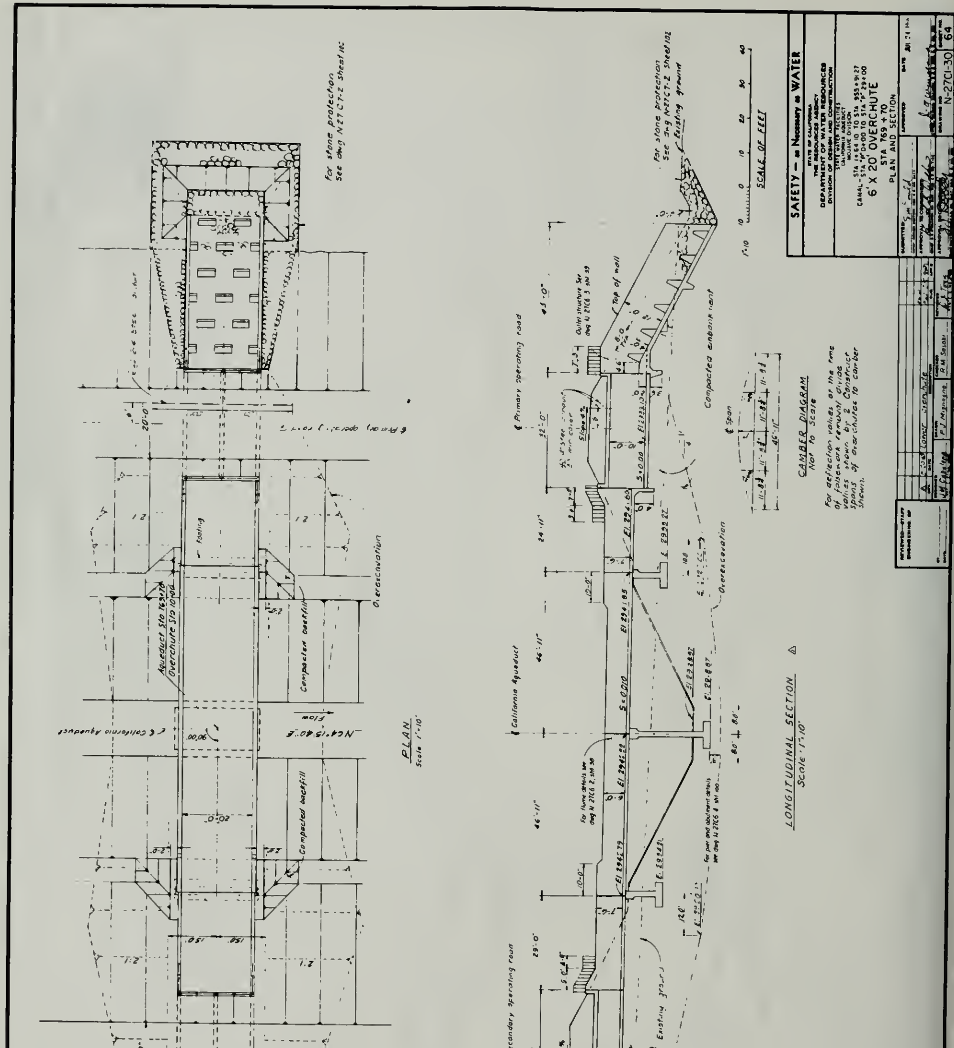


Figure 238. Overchute—Plan and Section

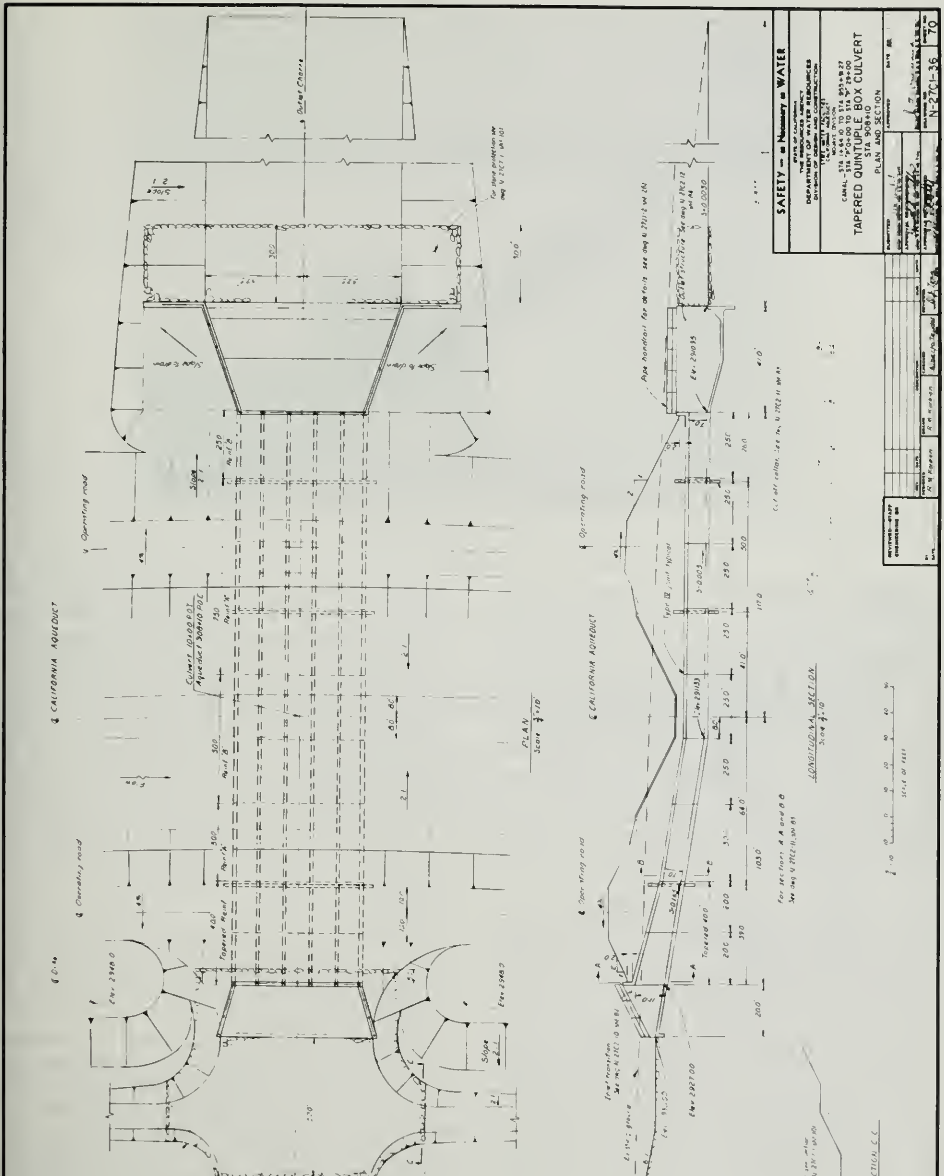


Figure 239. Box Culvert—Plan and Section

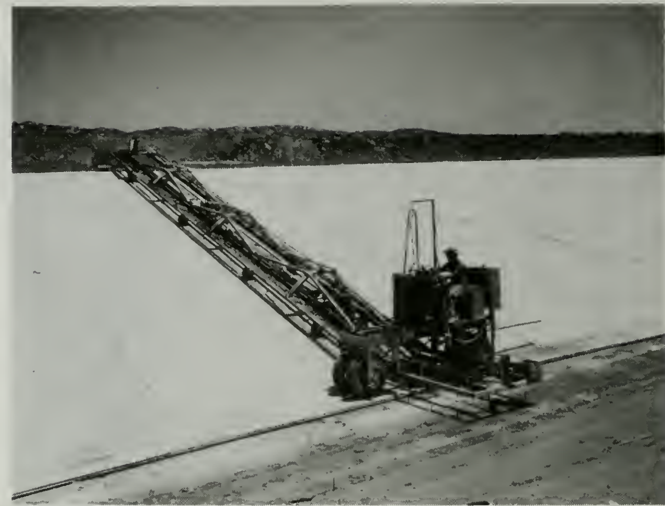


Figure 240. Sawing Transverse Contraction Joints

Pearblossom Pumping Plant to Los Angeles-San Bernardino County Line

Design. This 15.9-mile reach is the first of the three remaining major aqueduct construction contracts in the Mojave Division (Figure 241). The reach starts at the outlet of the discharge lines from Pearblossom Pumping Plant and is in canal section (Figure 242) except for two siphons, Tejon and Big Rock Siphons. Canal lining thickness was reduced from 4 inches or more to 3 inches, based primarily on the free-draining characteristics of the foundation soils. The slope of the canal is 0.00006 or 0.3 of a foot per mile and side slopes are 2:1. The canal prism is almost entirely in cut.

The first $\frac{3}{4}$ of a mile bears south; the rest swings east to the Los Angeles-San Bernardino County line. The area traversed for the first 5 miles, between Pearblossom Pumping Plant and Big Rock Creek (Figure 243), is of moderate relief through hills flanking the San Gabriel Mountains. Once across Big Rock Creek, the canal passes through an area of low relief cut by numerous shallow gullies. The vegetation is scattered brush and Joshua trees. The route area is undeveloped, with the exception of the area around Pearblossom and the Crystalaire residential and recreational development on Big Rock Creek.

There are 19 overchutes, 5 culverts, and 3 access bridges, but no turnouts. Prior to canal construction, the County of Los Angeles constructed five county road bridges.

Antelope Highway is the major state highway crossing this canal reach. The Antelope Highway bridge was constructed concurrently with the canal. To shorten the span of this bridge, the trapezoidal canal was transitioned to a rectangular channel.

The drainage facilities reflect the topographic relief. All five culverts are double, 54-inch, reinforced-concrete pipe and are in the hilly reach from Pearblossom Pumping Plant to Big Rock Creek. All of the overchutes are in the low relief terrain east of Big Rock Creek. The overchutes do not have riprapped inlet sections. Each outlet has a solid-wall stilling basin which leads to a graded, riprapped, energy-dissipator pad. Nine of the overchutes are 6-foot-high by 6-foot-wide, open, rectangular, reinforced-concrete sections; six are 6 feet by 10 feet; and four are 6 feet by 16 feet.

Brief intense rainstorms occur with peak intensities of over 5 inches per hour. Extensive use is made of 20-foot-high training dikes on the uphill side of the canal to route the large, sudden, sediment-laden floodflows to the overchutes.

The 611-foot-long Tejon Siphon is near Pearblossom Pumping Plant and carries the Aqueduct under 131st Street East (Longview Road). It is a double-barreled, 13-foot-inside-diameter, cast-in-place, reinforced-concrete pipe. This siphon is ungated but has provisions for upstream and downstream stoplogs. Each pipe barrel has a 16-inch blowoff controlled by a 12-inch butterfly valve. The siphon end transitions have provisions for a future third siphon barrel.

Big Rock Siphon (Figure 244) crosses Big Rock Creek and extends east past the Crystalaire residential and recreational development near Valyermo. Bi

Rock Creek drains a large area of the San Gabriel Mountains and is one of the major streams crossed by the Mojave Division. This 7,400-foot-long siphon is a single, cast-in-place, reinforced-concrete pipe 19.5 feet in diameter with a radial-gated inlet. A 15-foot-diameter pipe stub was installed at its inlet and outlet for future capacity increase. The Siphon has a 30-inch, blowoff, riser pipe which discharges at right angles through a short section of pipe controlled by 16-inch butterfly valves into a riprapped section of the Creek.

Another radial-gated check structure presently is being constructed in the reach between Big Rock Creek and the county line. This additional check structure is required to accommodate Phase II operational needs.

Leaving Pearblossom Pumping Plant, the first 5 miles of aqueduct foundation is in crystalline rocks, mostly quartz monzonite, but there is a 1,700-foot-long outcrop of hornblende gabbro. Beyond Big Rock Creek, the canal is in alluvium consisting of mixtures of clay, silt, sand, gravel, cobbles, and boulders. The major soil types are silty sands and well-graded sands with lenses of silt, sand, and gravel.

Construction. Only minimal clearing and grub-

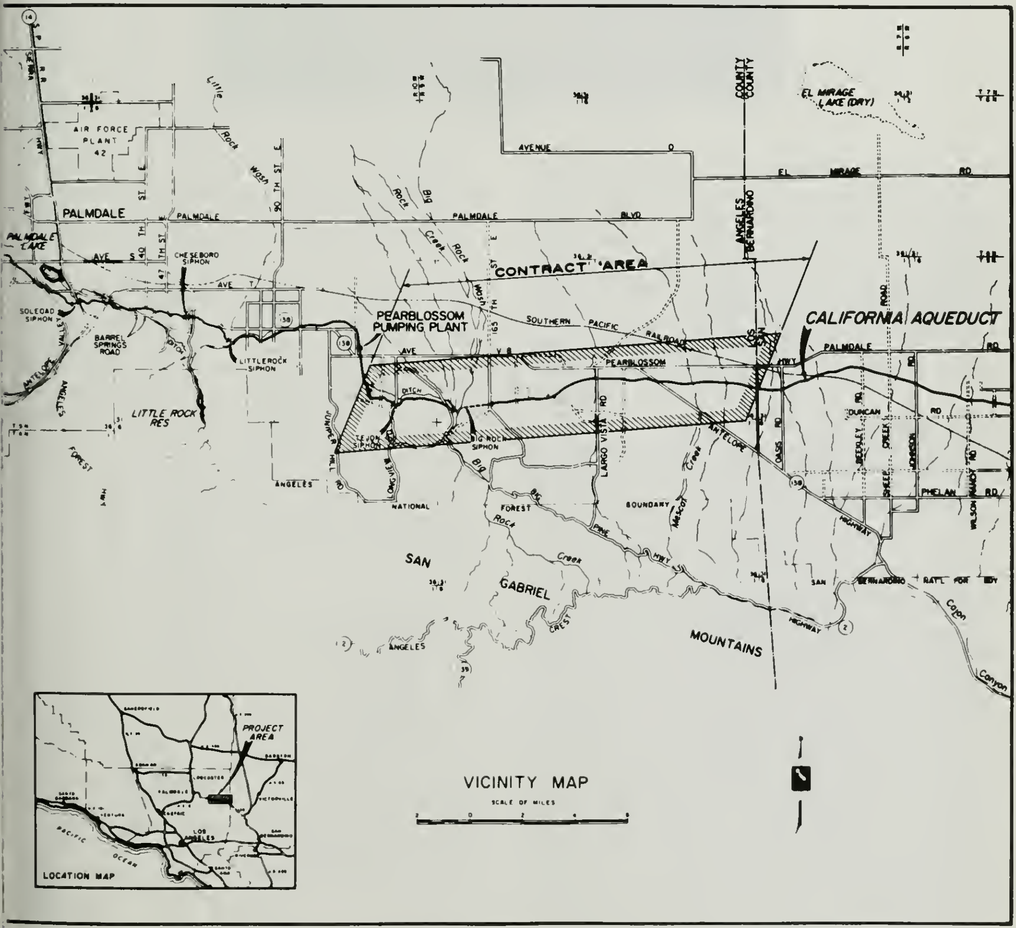


Figure 241. Contract Area—Pearblossom Pumping Plant to Los Angeles-San Bernardino County Line

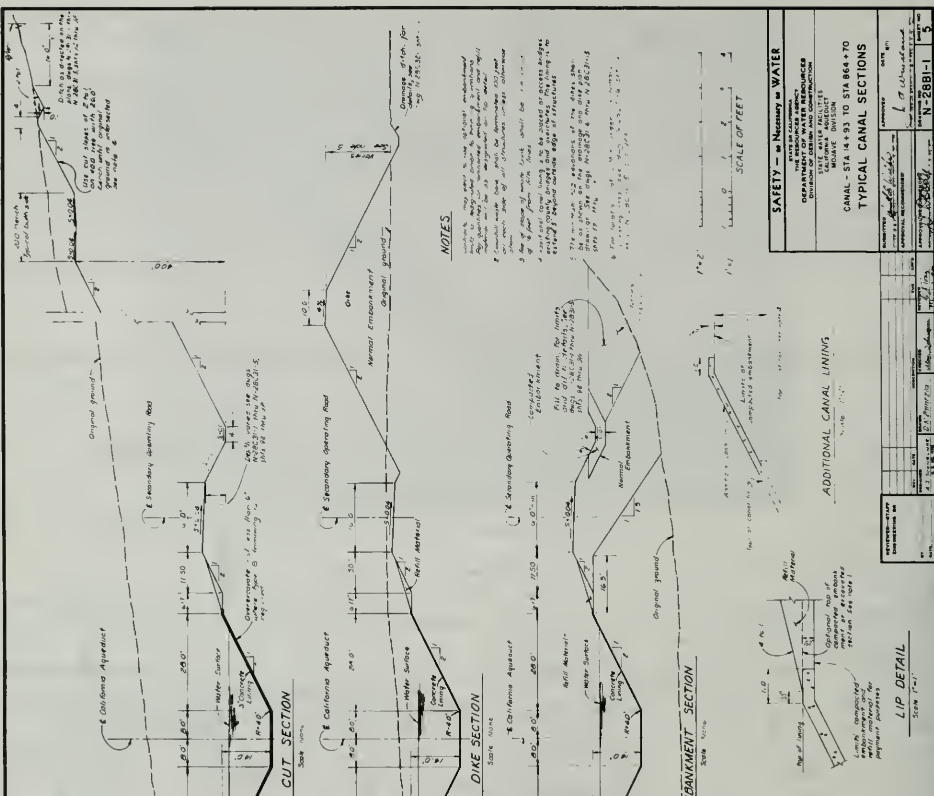


Figure 242. Typical Canal Sections—Pearblossom Pumping Plant to Los Angeles-San Bernardino County Line

Figure 243. Aerial View—Pearblossom Pumping Plant to Big Rock Creek

bing were required. Canal excavation was done by dozers and scrapers with some ripping required in the crystalline rocks between Pearblossom and Big Rock Creek. Cobbles and boulders encountered in the foundation were troublesome. A dragline sometimes proved to be the most feasible method of excavation.

Boulders and cobbles also were troublesome in the trimming operation. Considerable hand patching of the slopes was required.

Wherever overchutes were required, the contractor left earth plugs in the canal excavation to provide channels for runoff across the alignment. The merit of this precaution was proven when heavy runoff from rainstorms caused only minimal damage.

Trimming and lining operations were similar to previous reaches. A smaller size waterstop was needed because of the 3-inch-thick lining. The contractor elected to saw and seal the transverse construction joints. Paving of transitions and under bridges was done with a modified paver.

Each of the road bridge decks is supported by five prestressed-concrete girders. These girders were prefabricated and delivered to the job site. Using two cranes, one on each side of the canal, the girders for all of the bridges were installed in one day, and the bridge deck concrete then was placed and finished.

The foundation for Tejon Siphon mostly is in weathered and sheared quartz monzonite with some Recent alluvium. The trench was overexcavated from 6 inches to 2 feet. Material excavated from the siphon trench was stockpiled and used for compacted backfill. The pipe was cast-in-place, and the joints were filled with elastic sealant. Following backfill, the siphon satisfactorily passed leakage tests.

Excavation for Big Rock Siphon was mostly in sands and gravels but, in some areas, underlying weathered granitic rocks were found at or near invert elevation. Ripping as well as some blasting were required. Ground water was encountered in the pipe trench. Inflows during the dry months were 400 to 600 gallons per minute (gpm). During pipe casting, a French drain including perforated pipe was used for dewatering. When the contractor completed about 300 feet of pipe at the siphon inlet, 200 feet was backfilled for temporary relocation of Big Rock Creek and, in November 1970, a 25-year floodflow was passed without difficulty.

After placing circumferential reinforcing steel, longitudinal reinforcing steel for Big Rock Siphon was positioned and tied using a power-driven mandrel (Figure 245). Inside and outside movable forms were built and operated from a specially laid track (Figure 246). Numerous doors for inspection and vibration were built into these forms.

Water testing showed that leakage from this siphon exceeded specified maximum leakage of 11 gpm for a 24-hour period. The contractor then dry-packed, or pointed with cement grout, pipe joints that showed signs of leakage. He also pumped a liquid mixture of sodium silicate and additive into joints where leakage persisted and sealed especially stubborn joints with epoxy. After this repair work, although there were no remaining visible signs of leaks, leakage of 65 to 75 gpm was registered in tests. However, the Siphon was conditionally accepted on the expectation that the leakage would decrease to an acceptable amount as the material injected into the joints cured, and this reach of aqueduct was placed in operation. The Siphon was tested again one year after first water delivery and successfully passed the leakage test, and Big Rock Siphon was accepted unconditionally.

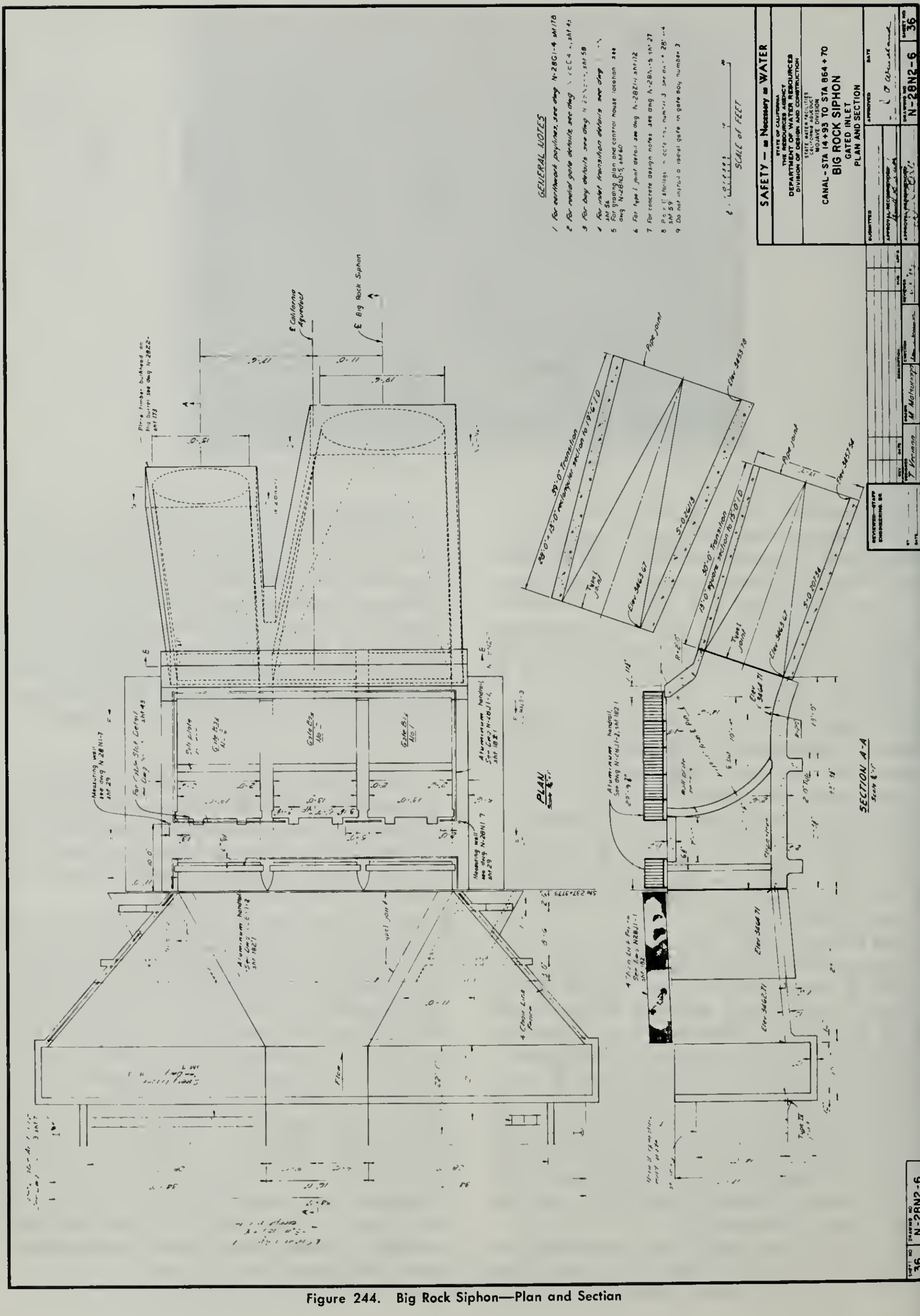


Figure 244. Big Rock Siphon—Plan and Section

Figure 245. Mandrel Used to Space and Tie Big Rock Siphon Rebars

Figure 246. Forms for Big Rock Siphon

Los Angeles-San Bernardino County Line to Mojave Siphon

Design. This 25-mile-long reach is the last canal reach in this division (Figures 247 and 248).

Terrain and geology are similar to that encountered from Big Rock Siphon to the county line. Most of the canal is underlain by alluvium, mainly silty sand. However, the last few miles of canal are through hills as the alignment closes on the Mojave River. This more rugged area required numerous sizable cuts and fills and one long siphon.

The route area has a grid of roads, mostly bladed dirt, that are on San Bernardino County's Master Road Plan. To accommodate these roads, 16 road bridges over the canal were included in the aqueduct construction contract. Prior to canal construction, bridges were built to carry State Highway 395 and Interstate Highway 15 over the canal. One railroad bridge also was built prior to aqueduct construction to carry the Southern Pacific Company's Cajon-Palmdale rail line over the canal 3 miles down-aqueduct from the beginning of the reach.

The Interstate 15 bridge is a double-box section which carries aqueduct water under the highway. Each box section is 16 feet high by 20 feet wide, and the double boxes are designed to carry the ultimate discharge including the 700-cfs, future, incremental, aqueduct capacity. The aqueduct construction contract included a double-radial-gated inlet and an outlet transition for this bridge. The transitions from canal to box section included provisions for inserting stoplogs.

The State Highway 395 bridge clear-spans the canal, and Southern Pacific Company's railroad bridge has two pile-supported piers within the canal waterway.

Antelope Siphon, the only siphon in this reach, crosses a broad valley south of Hesperia. It is 3,697 feet long and passes under the double-track Atchison, Topeka, and Santa Fe railway and Summit Valley Road. The railroad company constructed a temporary wood trestle while siphon construction was underway. A 36-inch, buried, high-pressure, gas line also crosses over the Siphon.

The Siphon, which ultimately will have three pipe barrels, was designed for four bidding alternatives: cast-in-place concrete pipe with steel liner, precast concrete pipe with and without steel liner, and prestressed-concrete cylinder pipe. Prestressed-concrete cylinder pipe was installed. The inside diameter was specified to be 126 inches; however, the contractor proposed supplying and installing 132-inch pipe at no additional cost to the State because his pipe manufacturer was set up to fabricate this size of pipe; this proposal was granted. In accordance with the specifications, the steel liner is No. 16 gauge; the amount of circumferential steel reinforcement varies with the depth of cover and hydraulic head on the pipe.

One barrel of the Siphon has been completed. A stubbed second barrel was built from the inlet extending under the railroad and bulkheaded for later completion. Presently, this second barrel is under construction for completion in 1975 as part of the Phase II work. Both the inlet and outlet transitions are designed for the ultimate installation of three siphon barrels. The inlet accommodates radial check gates which are being installed with the second barrel of the Siphon (Figure 249).

In addition to the check structure at Interstate 15, one other twin radial-gated structure was completed in Phase I between the county line and Interstate 15 (Figure 250). At the end of the reach, a twin radial-gated and stoplogged inlet for Mojave Siphon was constructed. This inlet is designed for two siphon barrels. The inlet for the east barrel of the Siphon was bulkheaded because construction of this barrel has been deferred until water demands on the Mojave Division justify its installation.

The many watercourses, although dry except during rainstorms, required 45 overchutes and 14 culverts. Overchutes range in size from a single, 6-foot-deep by 5-foot-wide, open, rectangular section to a triple-barrel overchute with a net waterway width of 36 feet and depth of 6 feet. Culverts range from a single 36-inch pipe to a four-barrel box, each barrel 8 by 8 feet. In addition, special drop inlets were constructed in the Hesperia area to drain runoff into the canal. These drop inlets are temporary features and will be removed when the County of San Bernardino implements its Master Drainage Plan in this area.

There are three turnouts in this reach: one for Mojave Water Agency (MWA) with a 30-cfs capacity; one for MWA with a 50-cfs capacity; and one for MWA, Desert Water Agency, and Coachella Valley County Water District with a combined capacity for all three agencies of 182 cfs. All have standard inlets and stoplogged and bulkheaded pipe outlets for later connection by the water contracting agencies. Two of the outlets (both for MWA) are single-barrel: one 30-inch- and the other 36-inch-diameter pipe. The third turnout (for the three water agencies) has two concrete pipes: one 36- and one 48-inch diameter.

The route is through Recent and Older alluvium nearly devoid of ground water within 50 feet of the canal invert. Most of the canal is in cut section with 2:1 cut slopes. In the rolling terrain from Antelope Siphon to Mojave Siphon, the strength of the Older alluvium permitted cut slopes of 1½:1. The canal configuration is the same as from Pearblossom Pumping Plant to Los Angeles-San Bernardino County Line, namely a 16-foot invert width, 2:1 side slopes, and a longitudinal slope of 0.00006.

Construction. Nearly ten million cubic yards of earth was excavated for the canal in this reach. The volume of compacted embankment was over one million cubic yards.

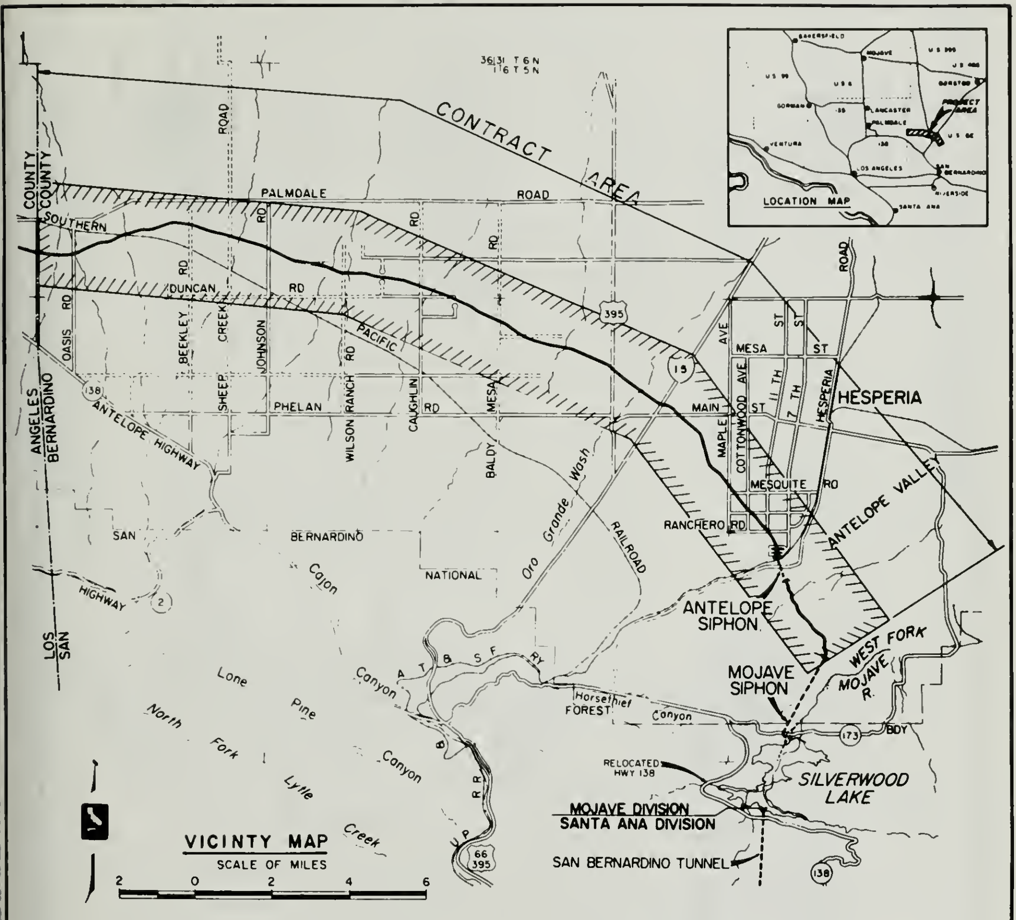


Figure 247. Contract Area—Los Angeles-San Bernardino County Line to West Fork Mojave River

The excavated areas were prewetted by a sprinkler system. Water was obtained from a source near Victorville. In nearly all soil tests, the in-situ soil-moisture content was well below optimum for compaction. In a few instances, ground water seeped from cut slopes into excavations but, generally, this did not cause construction difficulty.

Foundation conditions were suitable for spread footings for bridge piers, and abutments required only overexcavation and compacted backfill. Some of the culverts and transition sections for the overchutes also required overexcavation.

The contractor requested and was granted permission to substitute gravity walls for the specified buttressed transition walls for check structures, at siphon ends, and at the Interstate 15 bridge (Figure 251).

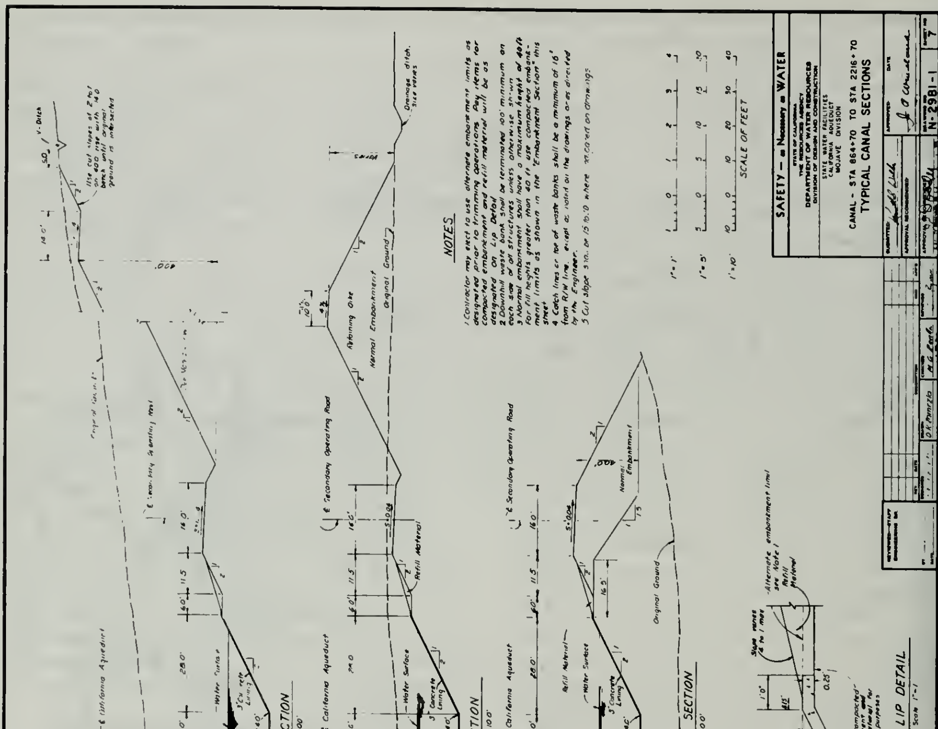


Figure 248. Typical Canal Sections—Los Angeles-San Bernardino County Line to Mojave Siphon

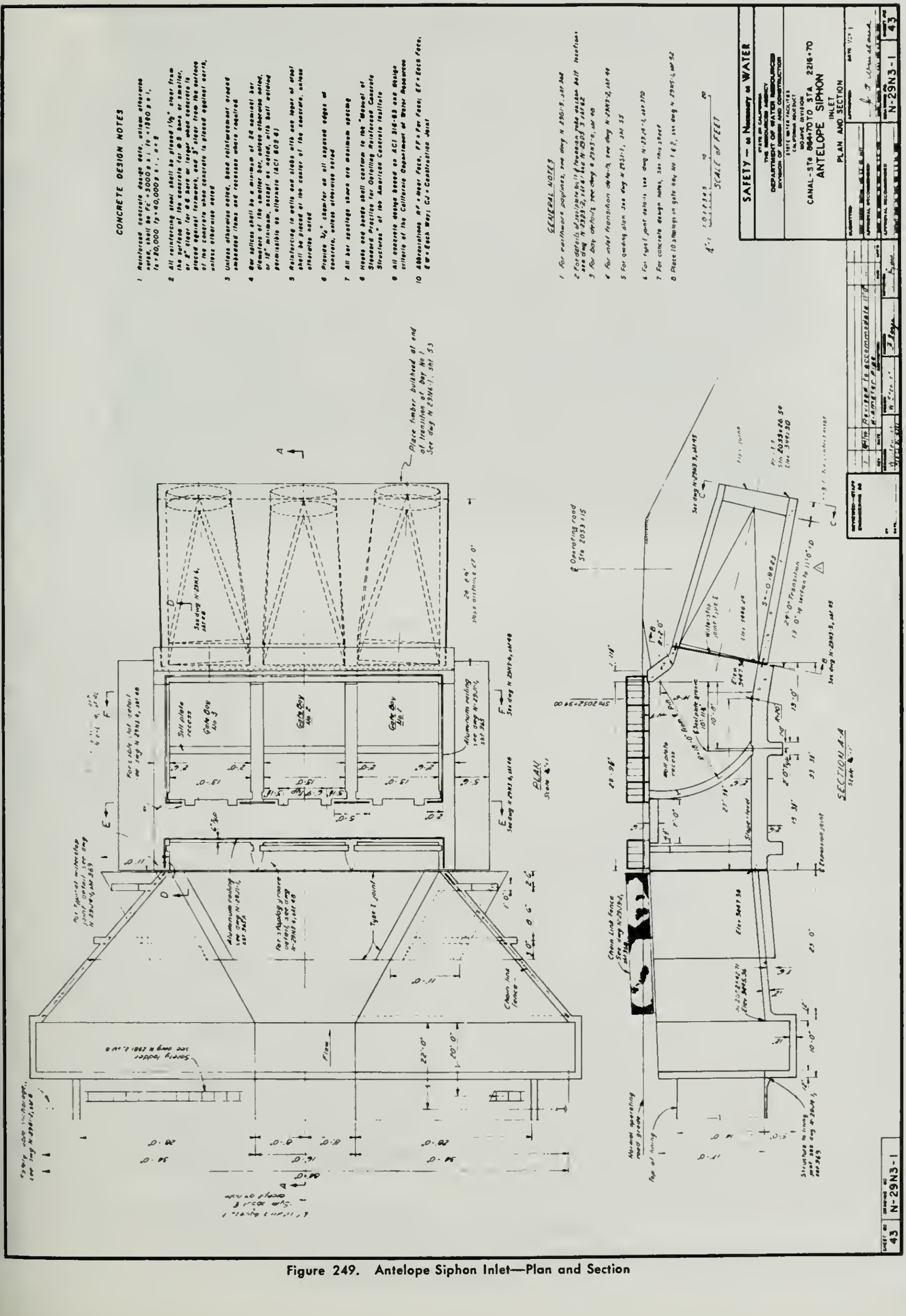


Figure 249. Antelope Siphon Inlet—Plan and Section

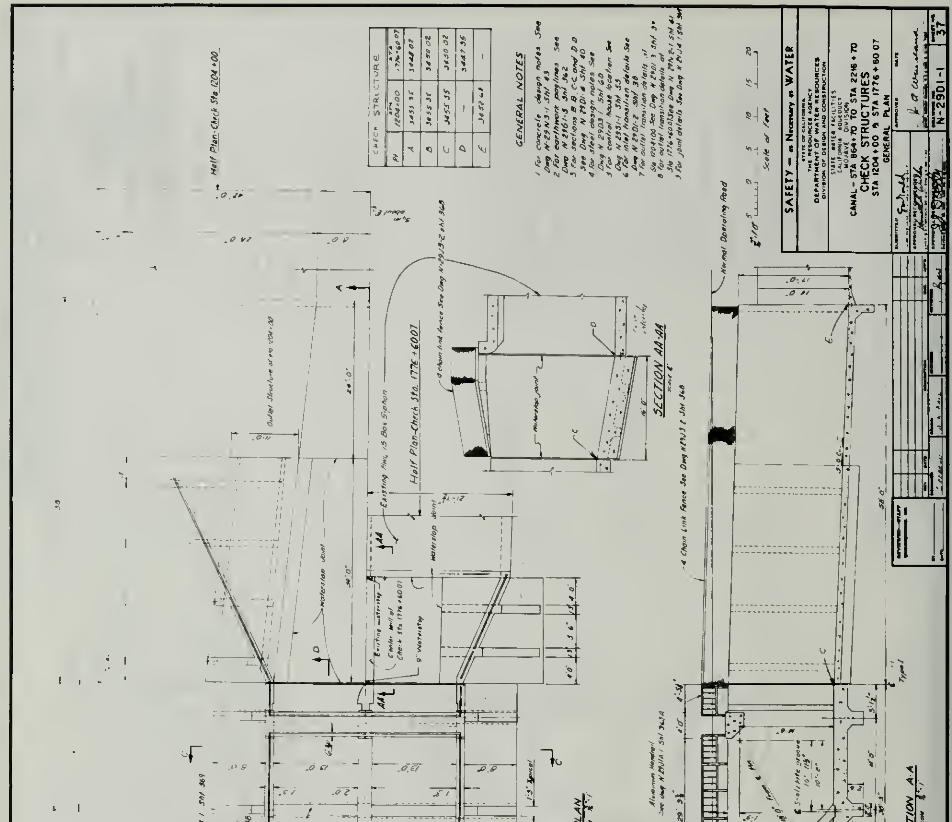


Figure 250. Check Structures—General Plan

Figure 251. Gravity Walls

Mojave Siphon

Design. Mojave Siphon is the final link of the Mojave Division conveyance facilities. The Siphon's total length from the check structure at the inlet to the outlet works at Silverwood Lake is 2.4 miles. It is designed to ultimately be a two-barrel siphon but only one barrel is now installed. The part described in this volume is 2.1 miles long, from the inlet (Figure 252). The extension of the Siphon past Cedar Springs Dam to its outlet into Silverwood Lake is covered in Volume III of this bulletin.

The Siphon crosses Summit Valley at the base of the San Bernardino Mountains 9 miles south of Hesperia. Summit Valley is drained by Horsethief Creek, an eastward-flowing stream tributary to the West Fork of the Mojave River.

The foundation for the Siphon comprises three geologic formations: Older and Recent alluvium and the Plio-Pleistocene Harold formation. The Harold formation, consisting of soft fluvial sands and silts, underlies alluvium at the site.

There were five bidding schedules for 126-inch-inside-diameter pipe: prestressed- and nonprestressed-concrete pipe, with and without a steel cylinder, and steel pipe. The structural design of the pipe was based on the pipe cover and internal pressure. The steel pipe was specified to be lined with either cement mortar or coal-tar epoxy.

There are four manholes and one blowoff in the pipeline. Manholes are 72-inch concrete pipes joined to the siphon pipe by concrete saddles and the manholes have hinged steel lids. A blowoff near the middle of the Siphon is an 18-inch, steel, riser pipe enclosed in a 72-inch concrete-pipe manhole. The blowoff is controlled by two 12-inch butterfly valves that discharge through a horizontal steel pipe into a riprapped open channel.

There is one turnout near the outlet end of the Siphon. This turnout is in a concrete vault and consists of a 30-inch steel-pipe riser with reducer to 18 inches and an 18-inch butterfly valve. The turnout is connected to a ranch irrigation system and replaces the ranch's prior diversion works on the West Fork of the Mojave River which were obliterated by construction of Cedar Springs Dam.

Construction. Bids were received for prestressed-concrete, steel, cylinder pipe only. The contractor, who also contracted for the contiguous reach upstream, subcontracted again with the same fabricator who supplied pipe for Antelope Siphon. Again, the substitution of 132-inch pipe was proposed at no extra cost to the State. This was approved with the stipulation that the contractor also provide, at no additional cost to the State, a reducer for joining the 126-inch pipe at the siphon outlet, previously installed by the Cedar Springs Dam contractor.

The siphon trench from the inlet (Figure 253) to the valley floor was excavated by motorized scrapers. Part of it was so steep it could only be excavated on downhill passes. Trench side slopes at 1:1 generally were satisfactory.

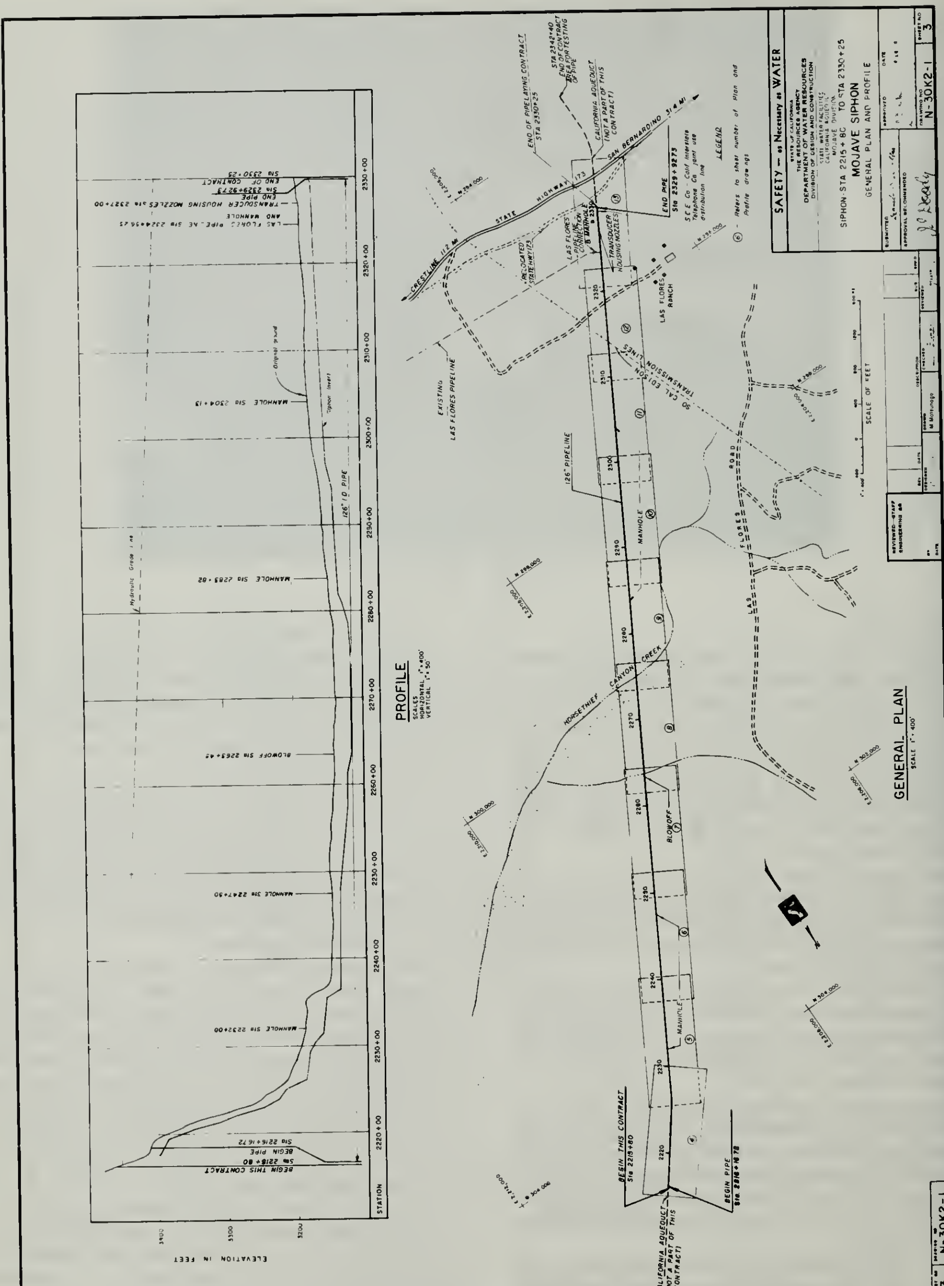


Figure 252. Mojave Siphon—Plan and Profile

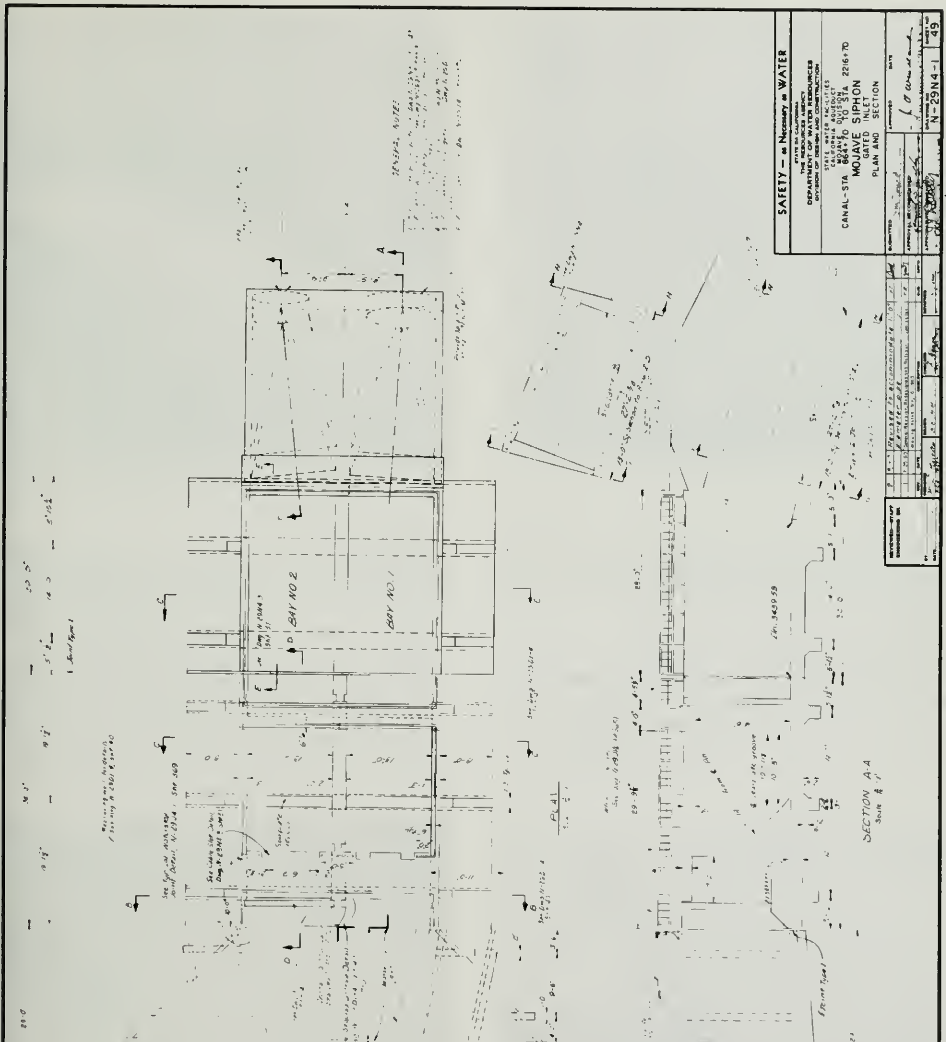


Figure 253. Gated Inlet—Mojave Siphon

On the valley floor and at the stream crossing, ground water was controlled by excavating a dewatering trench across the valley floor and 100 feet upstream from the Siphon (Figure 254). This trench was excavated through alluvium to an elevation below the top of the underlying Harold formation, and the inflowing water then was held below the level of the Harold formation contact by pumping.

The Siphon at the stream crossing was installed early in the contract to carry water pumped from the dewatering trench. Both scrapers and draglines were used to excavate the siphon trench, and a dragline was used to excavate the dewatering trench.

Twenty- and thirty-foot lengths of pipe was placed by the same equipment used to install Antelope Siphon, utilizing shorter pipe lengths on steeper grades (Figure 255).

At the stream crossing, the pipe was protected against flotation, in the event of backfill erosion, by 55 concrete saddles containing 1,298 cubic yards of concrete.

The last 100 feet of pipe was jacked into position from a concrete jacking foundation. The 132-inch to 126-inch reducer joining the Siphon to the outlet pipe was 4 feet long including two 8-inch, steel-sleeve, end sections. The sleeves were jacked into contact with the contiguous pipe sections after the reducer was positioned, then welded to the pipe steel cylinders and the reducer and sleeves encased in concrete.

The outside pipe joints were grouted. Then the Siphon was backfilled to a depth of one-quarter of the outside diameter of the pipe with consolidated granular material processed from local deposits. This material was consolidated by jetting and vibrating after placement by a number of methods, including end-dump trucks, a conveyor belt, front-end loaders, and clam shells (Figure 256). After placing the consolidated backfill, the trench was brought to final grade with uncompacted backfill. The interior joints then were dry-packed and pointed.

Cathodic protection was provided by establishing two shallow-well anode test stations, and bonding cables were exothermically welded across the inside joints and coated with coal-tar epoxy.

The pipe was given a 24-hour watertightness test. At the end of the test, only 368 gallons had been lost, which was less than 12% of the allowable loss.

Initial Operations

The aqueduct reaches leading to Pearblossom Pumping Plant were filled in late 1971 when project water became available. This provided water for Pearblossom Pumping Plant pump tests. Discharge water from the tests was ponded behind the first check downstream of the plant at the inlet to Big Rock Siphon. Water was introduced into the reaches downstream from Pearblossom Pumping Plant as each reach was completed. Project water was delivered to Silverwood Lake in 1973.

Because the remote control system and water-level reporting devices were not functional until well after initial filling and operation, much of the early aqueduct operation was done on-site. Close surveillance was maintained on embankments and structures.

Tehachapi Afterbay to Pearblossom Pumping Plant

The first pump at A. D. Edmonston Pumping Plant was started at 12:21 p.m. on October 7, 1971; and the first project water began flowing into the Mojave Division at approximately 9:45 a.m. on October 8.

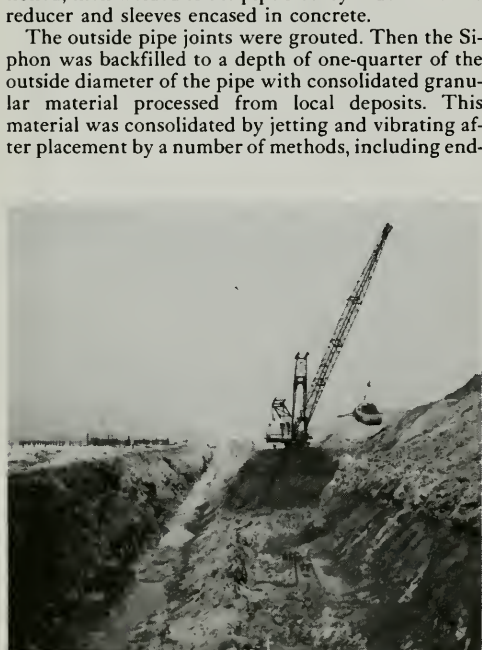


Figure 254. Dewatering and Pipe Trench Excavation

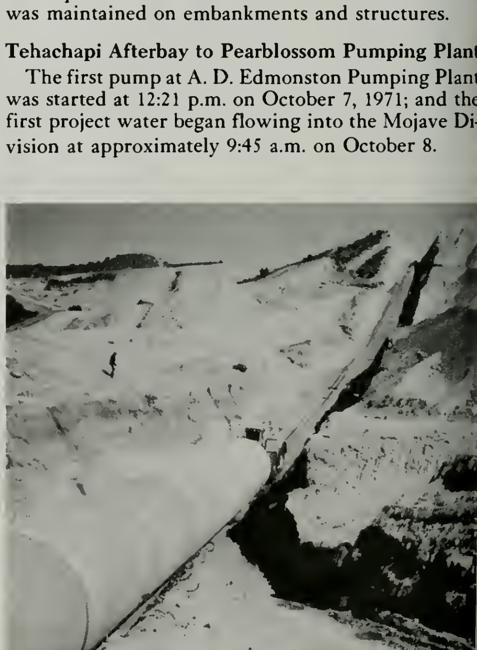


Figure 255. Pipemobile Near End of Winch Line

Figure 256. Vibrators (6-Inch) Mounted on Front-End Loader

Figure 257. Canal Break

Water flow was manually controlled sequentially at the check structures. When the water in a pool above a check reached normal depth of 12 feet, the next pool downstream was filled.

The reach between Willow Springs and Johnson Siphons (3.91 miles) was filled by approximately 1:00 p.m. on October 11, 1971. An aqueduct break (Figure 257) occurred in this reach between 9:00 and 10:00 p.m. on October 12, 1971, and 250 acre-feet of water escaped. The break was at the site of a box culvert about midway between the two siphons. The check gates at the siphon inlets were closed at the time and were effective in hydraulically isolating this reach and limited the volume of water spilled. Damage to adjoining properties was slight. Damage from the escaping water consisted of erosion immediately below the break and minor silting of fields and roads.

The break occurred in a full embankment section constructed on Recent alluvium between two cut sections. One hundred feet of canal was destroyed. All but two of the 25-foot-long sections of culvert were washed out (Figure 257).

The embankment and foundation conditions at the time of break were obliterated by the escaping water. A comprehensive investigation was made to determine the cause of the failure. It is probable a crack in the canal lining permitted seepage and piping of the embankment along the box culvert—conditions that progressively worsened until failure. There were no signs of sabotage. Excessive embankment settlement or foundation subsidence were not believed to have contributed to the failure. Also, there was no evidence that the San Fernando earthquake of February 9, 1971, with epicenter 19 miles away, affected the Mojave Division. At Fairmont Reservoir, the horizontal acceleration from this earthquake was 0.167g—probably the same acceleration experienced at the location of the break. The original embankment was designed for 0.5g.

The break area was excavated to solid foundation and the washed-out culvert replaced with a 78-inch reinforced-concrete pipe. Compacted embankment then was placed to full height and the canal reconstructed. Water was flowing through the canal again on December 20, 1971.

Divers inspected other reaches of the filled aqueduct and marked areas for closer observation where there was suspicion of cracked lining. As soon as feasible, these reaches were dewatered and repaired as necessary. All repairs were minor. Also, piezometer holes were drilled in some embankment sections to monitor ground water levels.

In the Littlerock area, about 1 foot of subsidence occurred in Recent and Older alluvium between 77th Street East and 82nd Street East. This was expected from geologic studies and operation of subsidence test ponds during the design phase. The subsidence was relatively uniform and within freeboard tolerances. The area has stabilized, and no further appreciable subsidence is expected.

Pearblossom Pumping Plant to Silverwood Lake

This reach of the Mojave Division was placed in service in 1972 as soon as Pearblossom Pumping Plant could deliver sustained flow. Water was introduced between checks in a sequential controlled operation. Close surveillance of culverts and embankments was maintained. Piezometer wells revealed no unusual rise in water levels and there was only minor settlement at drainage, check, and siphon structures.

CHAPTER XI. SANTA ANA DIVISION

Introduction

Role in the State Water Project

The Santa Ana Division serves water users along the southernmost reach of the California Aqueduct. Project water is transported from Silverwood Lake, through the San Bernardino Mountains, and past the San Bernardino-Colton-Riverside urban area to Lake Perris. Four water contracting agencies have direct turnouts from the Aqueduct in this division. Major water delivery is at Lake Perris from which distribution is made as far south as San Diego, California.

Aqueduct conveyance works in the Santa Ana Division are almost exclusively pressure conduits, operating by gravity flow. Power recoveries are made from the portion of this system upstream of Devil Canyon Powerplant.

A location map of the Santa Ana Division is shown on Figure 258, and a hydraulic profile is shown on Figure 259.

Hydraulic Function

The specific purpose of the Santa Ana Division is to convey project water from Silverwood Lake to Lake Perris using an elevation change of 1,775 feet. The division profile indicates approximately 1,400 feet of this elevation difference is utilized at Devil Canyon Powerplant.

Major water deliveries from Lake Perris are made to The Metropolitan Water District of Southern California. Other water deliveries are made directly from the Aqueduct through single turnouts to the San Gabriel Valley Municipal Water District, San Bernardino Valley Municipal Water District, San Gorgonio Pass Water Agency, and by three turnouts to The Metropolitan Water District of Southern California.

Geography, Topography, and Climate

The Santa Ana Division begins on the north side of the San Bernardino Mountains, where San Bernardino Tunnel bears almost due south through the mountains from Silverwood Lake to Devil Canyon. The south tunnel portal is in Devil Canyon, an important watershed for the City of San Bernardino. Leaving Devil Canyon Powerplant, the Aqueduct flows in a pipeline through urbanized San Bernardino County. The initial part of this pipeline route lies along State Street from which it runs parallel to the west branch of the Lytle Creek flood control channel; then, it crosses the Santa Ana River at the confluence of Warm Creek and rises through Grand Terrace to the Highgrove area of Riverside County, passes over a ridge line near Sugar Loaf Mountain, skirts the University of California at Riverside, and continues across Moreno Valley to Lake Perris. Lake Perris is 6 miles

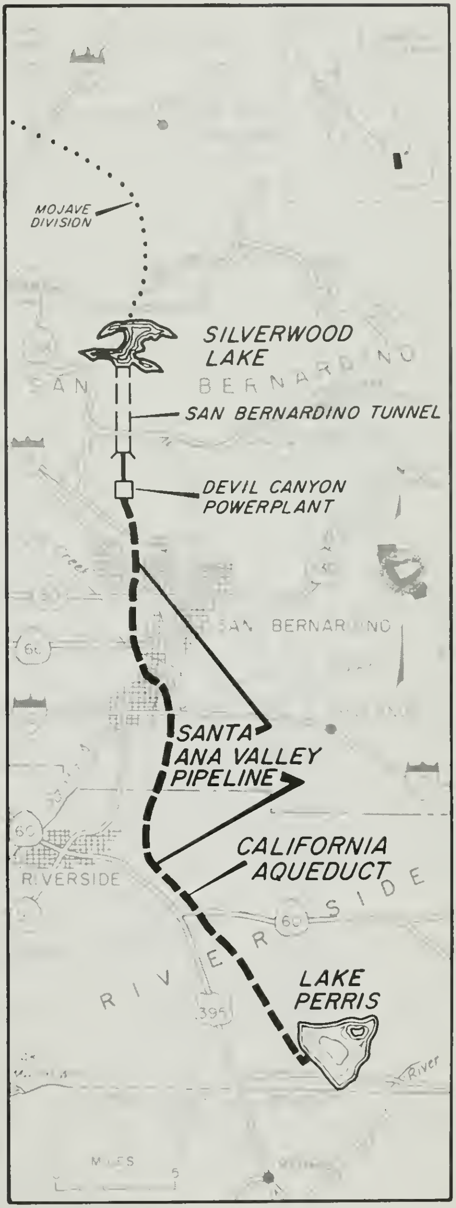


Figure 258. Location Map—Santa Ana Division

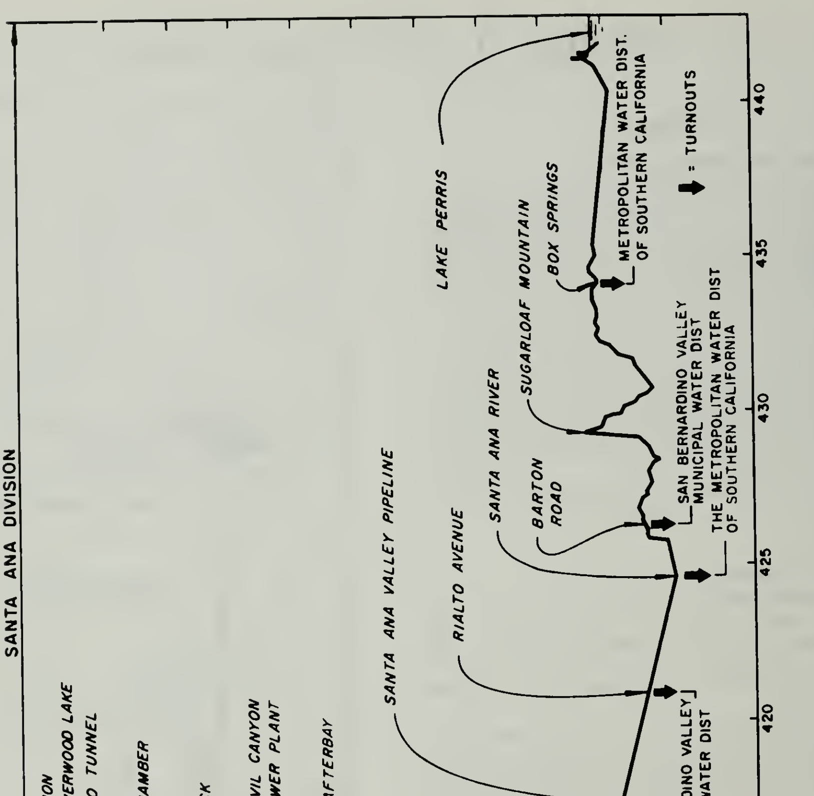


Figure 259. Hydraulic Profile—Santa Ana Division

northeast of the City of Perris and adjacent to Ramona Expressway.

Topography along this route ranges widely from the steep and rugged San Bernardino Mountains and moderately hilly areas near Sugarloaf Mountain to flat valley lands in the Santa Ana River Valley and the Moreno Valley approach to Lake Perris.

Typical climate in this reach of the Aqueduct is representative of a semiarid region on the edge of a desert. Usually, total rainfall is only 12 to 18 inches per year. Rain occurs mainly in the December to February period, and the greatest recorded monthly rainfall is 3 inches. Temperatures range from 55 to 95 degrees Fahrenheit in the summer and from 35 to 65 degrees Fahrenheit in the winter.

The prevailing winds are from the west. Infrequently, Santa Ana winds occur which are violent, dry, north winds sweeeping off the Mojave Desert through the mountain canyons. Often, these have sufficient force to topple power lines, fell trees, down antennas, and overturn trailers.

Features

In this chapter, only the conveyance works for San Bernardino Tunnel and Santa Ana Valley Pipeline are discussed in detail. Power plant and reservoir features are described only to the extent necessary to provide continuity. Devil Canyon Powerplant and penstock are discussed in Volume IV and Perris Dam and Lake Perris in Volume III, all of this bulletin. A statistical summary of Santa Ana Division conveyance facilities is presented in Table 24.

San Bernardino Tunnel

This is a 3.8-mile-long pressure conduit connecting Silverwood Lake with the Devil Canyon Powerplant penstock. The Tunnel has a circular cross section constructed to 13 feet in diameter. The unreinforced-concrete lining is approximately 24 inches thick. At the south or outlet portal, the last 425 feet of tunnel is reduced to 12.75 feet in diameter and has a ¾-inch steel-plate lining (Figure 260).

Water enters the Tunnel through a multiported intake tower located in Silverwood Lake, about 300 feet from the high-water shoreline. The tower is a vertical, reinforced-concrete, hollow cylinder 25 feet in outside diameter with 2.5-foot-thick walls and rises 157 feet above the tunnel invert to an operating deck. From this deck, inflow control equipment operates twenty-four 60-inch butterfly valves installed over circular openings inside the tower. Openings are grouped in six different tiers, spaced 18 feet vertically with four openings at each tier. Normally, the water level inside the tower will remain close to that of the reservoir to maintain head on the penstock. Access to the inlet tower from the shore is by a two-span plate-girder bridge 12 feet wide.

The Devil Canyon penstock surge chamber is installed over the San Bernardino Tunnel. It connects to the Tunnel through a 12.75-foot-diameter lateral shaft, intersecting 625 feet upstream of the south tunnel portal. This surge chamber is a vertical, circular, concrete tank 285 feet high and 37 feet in diameter. The center of the orifice plate at the bottom of the tank is 55 feet above, and 50 feet to the west side of, the Tunnel. Above the orifice, the tank extends 230 feet upward in the rock and 55 feet above the ground surface to the rim of this open-top tank, 340 feet over the Tunnel. Below the tank, the 12.75-foot-diameter shaft drops vertically to the elevation of the Tunnel where it bends 90 degrees to make the intersection.

Devil Canyon Powerplant and Penstock

This powerplant utilizes a 1,430-foot drop in the elevation of the Aqueduct, which makes it the highest head powerplant on the California Aqueduct. It now generates 120 megawatts, but provisions have been made for future expansion to 210 megawatts. The aboveground structure is approximately 100 by 150 feet in horizontal dimensions and houses two vertical impulse turbines and conventional generators. The plant structure will be enlarged and a third turbine unit installed in the future expansion.

The Devil Canyon penstock is 6,749 feet long, begins at the south portal of the San Bernardino Tunnel, and serves the two existing turbines. An upper bifurcation has been installed to accommodate a second parallel penstock which will serve the third turbine.

TABLE 24. Statistical Summary of Santa Ana Division

Aqueduct Reach	Type of Conveyance or Facility	Inside Diameter (Inches)	Capacity (Cubic feet per second)	Length (Miles)
San Bernardino Tunnel	Concrete-lined pressure tunnel	156	2,020	3.8
Devil Canyon Powerplant and Afterbay	Power generation	Data not applicable—See Volume IV		
Santa Ana	Concrete cylinder pipeline	108	469 and 444	20.5
		120	444	6.9
Lake Perris	Terminal storage	Data not applicable—See Volume III		

OPERATIONS

Manual on-site control or remote control from area control center, Santa Ana Field Division

Figure 260. Aerial View—South Portal of San Bernardino Tunnel and Devil Canyon Powerplant

A butterfly-type shutoff valve also is located between the bifurcation and the head of the penstock. Beyond this valve, the penstock drops in a steel ring-girder-supported pipe. Pipe diameters reduce in 6-inch increments along the penstock, from 9.5 feet at the top to 8.0 feet at the bottom. A lower bifurcation at the Powerplant routes water into the turbines.

The Powerplant discharges directly into Devil Canyon Powerplant Afterbay, an open pool with a capacity of 49 acre-feet. Water deliveries are made in four separate turnouts from the Afterbay and the pool provides regulation to reduce mismatches in flow between the plant and turnouts.

From the afterbay turnouts, water deliveries are made to San Bernardino Valley Municipal Water District, San Gorgonio Pass Water Agency, San Gabriel Valley Municipal Water District, The Metropolitan Water District of Southern California, and the Department of Water Resources' Santa Ana Valley Pipeline.

Santa Ana Valley Pipeline

Santa Ana Valley Pipeline is the aqueduct connection between Devil Canyon Powerplant Afterbay and Lake Perris and is 28 miles long. The first $20\frac{1}{2}$ miles is 9 feet in diameter and the final $7\frac{1}{2}$ miles is 10 feet in diameter, all in prestressed-concrete cylinder pipe. Maximum pressure head in this pipe reaches 950 feet of water at the Santa Ana River crossing (Figure 261).

Figure 261. Installing a Typical Reach of the Santa Ana Valley Pipeline

Lake Perris

The terminus for the Santa Ana Division is Lake Perris. This reservoir stores 131,452 acre-feet. Perris Dam is a zoned earthfill 128 feet high and approximately 11,600 feet long.

Geology and Soils

Geology

The Santa Ana Division starts on the north side of the San Bernardino Mountains, a mountain block that extends from Cajon Pass eastward for a distance of about 60 miles to San Gorgonio Pass. San Bernardino Tunnel passes through these mountains. San Andreas fault borders the mountain block on the south side, and a series of westerly trending faults border the Mountains on the north side. In the vicinity of the conveyance system, the mountain range is composed of igneous and metamorphic rocks of late Mesozoic Age. Erosional remnants of Tertiary continental sediments occur in a few small areas on a crest of the range and also lap onto northerly flanks of the mountain block.

South of the Mountains, Santa Ana Valley Pipeline crosses a broad valley deeply filled with alluvial detritus derived from adjacent mountains. In the vicinity of Sugarloaf and Box Springs Mountains, granitic rocks protrude through alluvium, forming low hills. From Box Springs Mountains to the end of the California Aqueduct at Lake Perris, the Pipeline crosses a broad valley filled with alluvium and underlain by granitic rocks.

The Aqueduct crosses two major faults in this division. The San Andreas fault is crossed just downstream from Devil Canyon Powerplant, and the San Jacinto fault is crossed twice—once at Lytle Creek and again at Warm Creek. Between these two latter fault crossings, the Santa Ana Valley Pipeline closely parallels the San Jacinto fault. The frequency of earthquakes along San Jacinto fault probably is as great as any fault in California. Strong earthquakes have been reported historically on both the San Jacinto and San Andreas faults in the vicinity of San Bernardino. Surface fault rupture may have occurred along the San Andreas fault as far as San Bernardino in the 1857 Fort Tejon earthquake, because scarps along the fault indicate movement has occurred in recent geologic time. Similar but smaller fault scarps also are seen along the San Jacinto fault. Numerous other smaller faults cross San Bernardino Tunnel through the San Bernardino Mountains. Faulting in the San Bernardino Mountains increases to the south as the San Andreas fault is approached.

Both San Andreas and San Jacinto faults were identified by detailed mapping in the vicinity of fault crossings. Two fault quadrilaterals were established, one at Rialto and one at Colton, to monitor fault creep along the San Jacinto fault. Another quadrilateral, to monitor creep along San Andreas fault, was located at the mouth of Devil Canyon. A strong-motion seismograph and an array of seismoscopes also were installed at Devil Canyon to monitor ground motion. This system has been used continuously since completion.

Since the Aqueduct is exposed to damage during earthquakes from ground motions or from fault displacements on both San Jacinto and San Andreas fault systems, earthquake-resistant design criteria were developed to give optimum protection from these seismic hazards.

Some small mountain communities in the San Bernardino Mountains were located over the San Bernardino Tunnel alignment and obtained their water supply from horizontal wells and springs which tap water stored in fractures in crystalline rocks. It was anticipated that driving the San Bernardino Tunnel could drain the mountain block and dry up these water sources. A program to measure springs and wells in the vicinity of the tunnel alignment started in 1953 and was continued during construction and into the the postconstruction period of the Tunnel. This program established a series of records on local water supplies to determine, by comparison of flows, if tunnel construction changed the supply in any way.

Preconstruction studies were made of the amount of water that might be drained from the rock formation during the tunneling so that precautions could be taken during construction to avoid impairment of the water supply. Precautionary procedures were selected to probe by drilling ahead of the tunnel heading. When large flows of water were encountered, the formation was grouted to seal off this flow before advancing the heading. Several large inflows of water were encountered, but this exploration procedure worked satisfactorily and ground water supplies were not seriously impaired by the Tunnel.

Soils

Soils in the San Bernardino Mountains consist of weathered crystalline bedrocks and some soft sedimentary rocks on the north slope. When excavated, these materials usually range from silty sand to sandy clay mixtures. South of the Mountains on the valley floor, soils along the Pipeline are primarily silty sands. Between Devil Canyon and Sugarloaf Mountain, considerable deposits of sand and gravel are found. East of Sugarloaf Mountain, alluvial soils are almost entirely silty sand but include some sandy clays.

Design

Overall design criteria for the Santa Ana Division were the same as for similar parts of the State Water Project. However, certain local factors required special attention. For the Tunnel, the most important factor was an expectation that large inflows of water would occur during mining. For pipeline reaches, factors influencing design were the exceptionally high pressure heads in the pipe, geological faulting, potential seismicity risks of the route through an area of high population density, and alignment conflicts from intensive urban-type development.

Hydraulics

The hydraulic gradeline for the Santa Ana Division begins at elevation 3,355 feet in Silverwood Lake and ends at elevation 1,580 feet in Lake Perris. This large elevation change requires the Aqueduct to be considered in two reaches for hydraulic purposes: first, a power-generating reach, and second, a pipeline reach (Figure 262).

In the power-generating reach, the elevation, size, and hydraulic gradient of the San Bernardino Tunnel were selected to minimize the volume of dead storage in Silverwood Lake, minimize down-surge effects on the Devil Canyon penstocks, and attain the greatest value of power recovery. Devil Canyon Powerplant Afterbay also was provided for a limited amount of regulatory storage.

Two hydraulic design concepts were considered for the Santa Ana Valley Pipeline reach. A static head concept used flow controlled by valves along the Pipeline, while a falling hydraulic grade concept eliminated in-line valves and required only inlet control at the Afterbay, with a free outfall into Lake Perris. Both concepts had ample precedents and were studied in detail during the preliminary design stage.

The static head concept would allow flow control and hydraulic isolation of the Pipeline into reaches by in-line control valves and would permit overall flow control at the downstream end of the Pipeline at Lake Perris. This concept required that the Pipeline be designed for full internal pressure imposed when flow was stopped and water backed up from outlet to inlet. The advantage of static head design was flexibility to operate parts of the Pipeline with minimal contraints, but this had to be judged against relatively large capital expenses for high-pressure valves and high-pressure reaches of pipeline.

Under the falling hydraulic grade concept, the Pipeline would be designed only for the net internal pressure during gravity-flow conditions. Design pressure in this case was less than static pressure by the amount of energy losses in the Pipeline. This was equal to 240 feet of water on differential head between the Afterbay and terminus.

More operational constraints had to be placed on a pipeline designed for the lower head of a falling hydraulic gradient than one designed for full static head. One of these constraints was that flow control could be located only at the pipeline inlet at Devil Canyon Powerplant Afterbay. Another constraint was that future in-line valves or valves at the pipeline terminus outlet at Lake Perris could not be used because malfunctions in valve operation could raise the internal pipe pressure above design pressure.

The falling hydraulic grade concept finally was selected for the Santa Ana Valley Pipeline based on the judgment that reduced capital cost justified reduction in operational flexibility.

San Bernardino Tunnel

Geologic investigations indicated the entire Tunnel would be in granitic rock formations containing a few thin fault zones. Ground water inflows and associated construction hazards were expected because of the previously mentioned ground water program to monitor springs and wells in the area above the Tunnel.

Physical data from tests made before construction showed typical rock conditions as:

Dynamic Modulus of Elasticity	6.01 to 12.20 $\times$ 10^6 pounds per square inch (psi)
Static Modulus of Elasticity	4.12 to 8.42 $\times$ 10^6 psi
Poisson's Ratio	0.16 to 0.44
Ultimate Compressive Strength	13,550 to 16,360 psi
Ultimate Tensile Splitting Strength	1,044 to 1,376 psi
Bulk Specific Gravity	2.71 to 2.77

The maximum rock cover on the Tunnel was assumed as 2,200 feet.

Santa Ana Valley Pipeline

The Aqueduct from Devil Canyon Powerplant Afterbay to Lake Perris was designed as high-pressure pipeline using the falling head concept previously described. Minimum design pressure head was 80 feet and maximum 950 feet. The design did not consider hydraulic transients. Four contract reaches were selected for this free-flowing conduit. Turnout locations and delivery schedules required a design capacity of 469 cubic feet per second (cfs) from Devil Canyon Powerplant Afterbay to Barton Road turnout and 444 cfs beyond to Lake Perris. Pipe diameters were established at 9 and 10 feet. Combined air release-vacuum valves were installed at crests on the pipeline profile, and access manholes were spaced at 4,000-foot maximum intervals. Four major pipeline blowoffs were installed using special designs because releases were required at over 400 feet of differential pressure head through the blowoff valves.

Turnouts were designed at pipeline pressure from 90-degree tee sections, controlled by electric-motor-operated, 24-inch, spherical, shutoff valves located in buried concrete vaults. Vaults were vented and equipped with segmented, removable, top slabs for access to the equipment. All turnouts were wired for metering and telemetry of flow data to a central station.

Aqueduct alignment was selected to result in the shortest overall pipeline commensurate with geologic and right-of-way restraints and for ease of construction and operation. Simplification of construction was especially important because the Pipeline had to traverse long reaches along city streets or county roads. Operational criteria required blowoffs

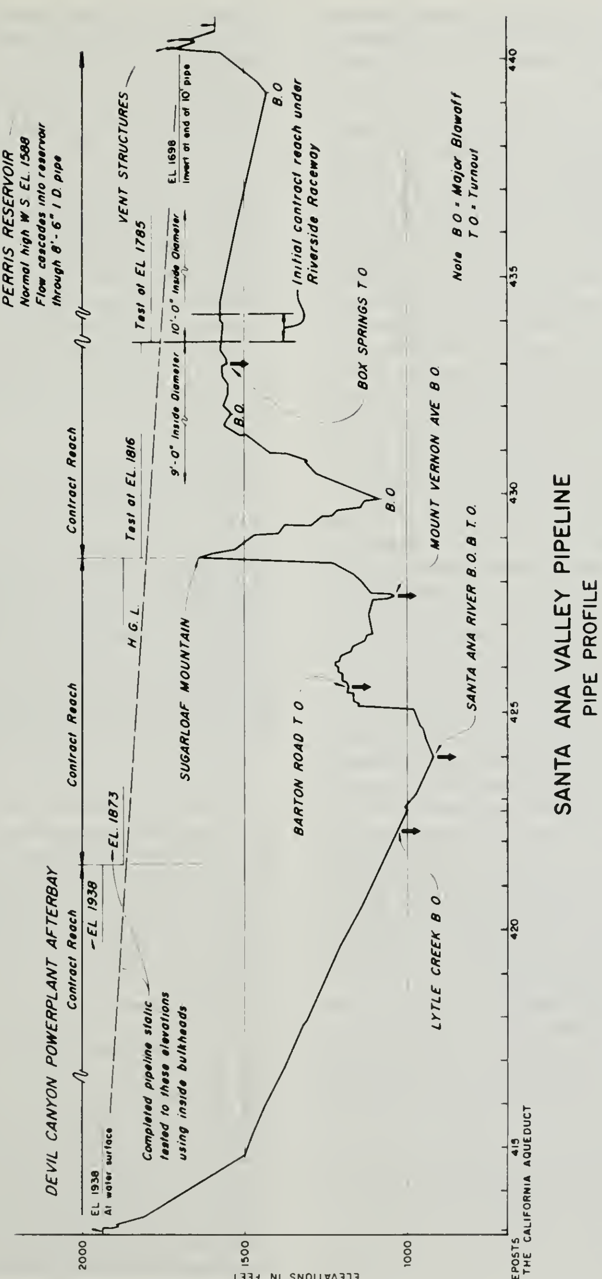


Figure 262. Pipe Profile

located at sites where rapid disposal of water could be made if a break occurred. Necessary alignment changes were made by long radius curves to eliminate the need for anchorages.

Pipe was laid in trench excavation which typically allowed a nominal 7-foot clear cover over the pipe, although at certain places maximum cover approached 25 feet. Effects of such earth loads on the pipe barrel were computed by coefficients typical for buried conduits. Live loads on the buried pipe usually were much less than static earthload from backfill. Live loads used for design were equal to AASHO H-20-S44 for roadways and Cooper E-72 for railroads. For pipe installation at major arterial roads, through levees, and under railroad tracks, mined construction was required in which an oversized casing supported the excavation around the enclosed aqueduct carrier pipe. Annular void spaces were filled with cement-mortar grout. In city streets, pipe was laid in shored trenches, and only 660 linear feet of excavation was allowed to be open at any time. Surplus excavation was stockpiled or hauled to designated spoil sites.

Backfill was specified to provide optimum support for a variety of pipe shells and to support finished surface conditions. Where finished surface pavements were governed by aesthetic considerations, compacted backfill was placed to finished ground subgrade. Elsewhere, consolidated granular backfill was placed to the top of the pipe and the remainder of excavation replaced to a nominal density for the area.

Bids were invited for construction of alternative pipelines which used either steel or prestressed-concrete cylinder pipes. In all cases, only the concrete-pipe alternative was used by the successful low bidder.

Steel plates for the steel-pipe alternative conformed to ASTM Specification A572, Grades 30, 42, and 50, and were finished to ASTM Specification A20 for pressure-vessel instead of structural provisions. This steel specification, new at the time of design, provided strength and ductility properties most suitable for this highly stressed pipe. It was substituted for ASTM Specification A283 or A285C. Working stresses were established as 0.65 yield point or 0.33 ultimate. Charpy impact tests also were required to show a minimum of 15 foot-pounds at 32 degrees Fahrenheit.

Stiffener rings were specified for steel pipes. These were 8-inch by $\frac{3}{4}$-inch hoops, spaced nominally at 12 feet along the pipe. Maximum stiffener working stress was 27,000 pounds per square inch (psi). Compacted backfill was designed in conjunction with optimized stiffener spacing and placed to a minimum of 2 feet over the top of the pipe.

Four alternative steel-pipe joints were allowed: all welded bell and spigot joints with single or double vee welds, butt-welded joints, bell- and spigot-type joints using Carnegie shapes with rubber "O"-ring gaskets, and sleeve-type couplings.

Corrosion protection for the steel pipe was specified to include continuous $\frac{1}{2}$-inch cement-mortar linings and coal-tar-enamel wrapped coatings, complete electric bonding across nonwelded joints, and corrosion test stations to measure the insulation effectiveness of this system.

Concrete-pipe alternatives specified prestressed-concrete cylinder pipe designed by the Department. This design conformed to AWWA Standard C301, except as modified by department criteria. Typically, this consisted of a pipelike unreinforced-concrete core which was wrapped uniformly by special steel wire at close spacing and high tension to provide an overall compressive force in the pipe wall under all service conditions for internal and external loading. All cores included an encased and continuous No. 16-gauge steel cylinder which provided a watertight membrane between pipe joints and added minor strength properties to the pipe. Wrapped cores were completed as pipe units by covering the prestressing wires with a minimum of $\frac{3}{4}$ of an inch of high-strength Portland cement mortar.

Core thicknesses were based on pipe-industry standards. Since pipe cores were to be mass-produced in concrete forms, they required minimum dimension changes. Strengths of constant-thickness cores were varied by changing the strength of the concrete mix.

External load effects on completed pipes were determined by thrust and bending moment coefficients obtained from a bulb-type distribution of earth pressures on the outside of the pipe. Pipe bedding directly influenced the external load coefficients. An optimized condition was obtained by consolidated granular bedding material, designed to provide 90 degrees of pipe invert support but specified for construction as a 120-degree cradle in granular bedding.

For the cores, ultimate concrete compressive strength at 28 days (f'_c) varied from 4,500 psi to 7,000 psi. Allowable compressive stress in a core was 0.45 f'_c at normal loading conditions from design pressure and backfill and 0.60 f'_c for rare overload conditions, which included AASHO H-20 live loading at the ground surface over an empty pipe. Concrete unit tensile stresses, caused locally by bending effects during external loading on an empty pipe, was allowed in the core at $7.5\sqrt{f'_c}$ for normal loading and $10\sqrt{f'_c}$ for overload.

For final design, all 9-foot-diameter pipes used 10-inch cores, except on reaches near the Santa Ana River where a 25-foot depth of backfill cover required design of a 12-inch core. On the 10-foot-diameter reach, a $9\frac{1}{4}$-inch core was used.

Pipe joints were designed using common $\frac{3}{8}$-inch, mild-steel, bell rings and Carnegie-shape spigot rings. The latter shape provided the groove for $\frac{3}{4}$-inch, rubber, "O"-ring gaskets compressed to make a watertight seal when engaged by the bell ring in the adjacent pipe. Both steel components of the joint had to be rolled, butt-welded, and cold-expanded to a standard circumference because of the close tolerances require to fully compress the "O"-ring gaskets. Bells and spig ots were welded continuously to the steel cylinders.

In typical bell and spigot joints for these pipes, the shank of the spigot ring formed a cylinder which was unsupported for about 3 inches of its length. This ring was exposed to full pipeline pressure and, as a design precaution, the steel specification for spigot rings was changed to ASTM A572, Grade 50, when internal pressure exceeded 425 feet.

Prestressing wire was specified equal to either ASTM A277, Class II, or ASTM A648, Classes II or III. It was wrapped on the core while under a tensile force equal to ¾ of the minimum ultimate strength of the wire. Wire sizes included No. 8, No. 6, ¼-inch, and $^{5}/_{16}$-inch. Since the prestressing force provided by the wrapping was determined directly by the service loads on the pipe, specifications allowed alternative designs which furnished the required pipe core compression from either single or double wrapping. The second layer of wrapping was necessary on pipe for higher pressure heads because there was a minimum practical limit for spacing of the wires. Also, the wire diameter was limited by physical capacity of the pipe manufacturer's equipment to apply the wrap at the required tension.

The dense mortar coating applied to the outside of the wrapped cores protected the wire from damage and furnished an alkaline atmosphere to minimize corrosion. This mortar also added to the strength of the completed pipe since its strength was equal to or greater than that of the core. For control during fabrication, a specification was included that the coating be checked continuously by absorption tests. These tests were used as a parameter to evaluate consistency in the density of the coating.

Special Conditions

Alignment of Santa Ana Valley Pipeline. A major design consideration in the Santa Ana Division was selection of an alignment through urban areas of San Bernardino, Colton, and Riverside with least exposure to disruption and damage in the event of a major earthquake. In urban areas and especially for high-pressure reaches, alignment was restricted to streets and along flood channels. In rural and agricultural areas, an alignment generally along the shortest route was selected.

Inlet Water Surface for Santa Ana Valley Pipeline. Preliminary studies showed that the optimum economical location for Devil Canyon Powerplant was south of the nearby recent trace of the San Andreas fault. Since a plant at that location required the high-head penstocks to cross the fault, it was eliminated and another site selected north of the San Andreas fault. This second site raised the elevation of Devil Canyon Powerplant and Afterbay by 200 feet above the first site. Decreases in generating head at the Powerplant were compensated by a corresponding increase in the hydraulic gradeline at Devil Canyon Powerplant Afterbay, which permitted a reduction in the diameter of pipe used for the entire Santa Ana Valley Pipeline.

Aseismic Design of Santa Ana Valley Pipeline. Alignment of the Santa Ana Valley Pipeline traverses a highly seismic area, particularly in the first 9½ miles between Devil Canyon Powerplant (elevation 1,920 feet) and Warm Creek (elevation 960 feet). Two large active faults, the San Andreas and San Jacinto, which lie within this region have been described previously. This reach also is in the City of San Bernardino and passes close to present and future commercial and residential lands.

According to department consultants, a major earthquake may be expected on either of the above faults during the life of the State Water Project. They advised that a displacement of 10 feet horizontally and 5 feet vertically could occur in a period of about five seconds with ground accelerations of 0.5g horizontally and .33g vertically. Displacements would be most likely along recent fault breaks. To minimize effects of fault displacements, a pipeline route was selected with the shortest path across the faults.

It was considered impossible to design a pipeline completely resistant to the potential displacements at fault crossings. Therefore, aseismic design criteria were adopted to minimize damage if seismic activity caused leakage from the Pipeline. These pipeline design criteria included the following provisions for passing through traces of major faults: (1) the pipeline route would cross the fault at the fewest locations, with the shortest pipe length, and within the least developed urban areas; (2) this route also would follow, as much as possible, existing flood control channels or developed drainage channels; (3) high-capacity, pipeline, blowoff structures would be installed to quickly dewater the Pipeline from at least four locations; (4) extra widths of right of way would be provided to remove private property farther from the aqueduct conduit; (5) joints in the Pipeline would be made with articulation near fault crossings to allow greater pipe flexibility; and (6) existing seismographic and fault-movement monitoring stations would be retained and instrumentation installed for continued monitoring of the completed structure.

Construction

General information about the major contract for the construction of the Santa Ana Division is shown in Table 25. These contracts include the San Bernardino Tunnel and four pipeline reaches extending from Silverwood Lake to Lake Perris.

Construction supervision was administered from the project office located in Palmdale. Field offices were located at Devil Canyon Powerplant and Perris Dam.

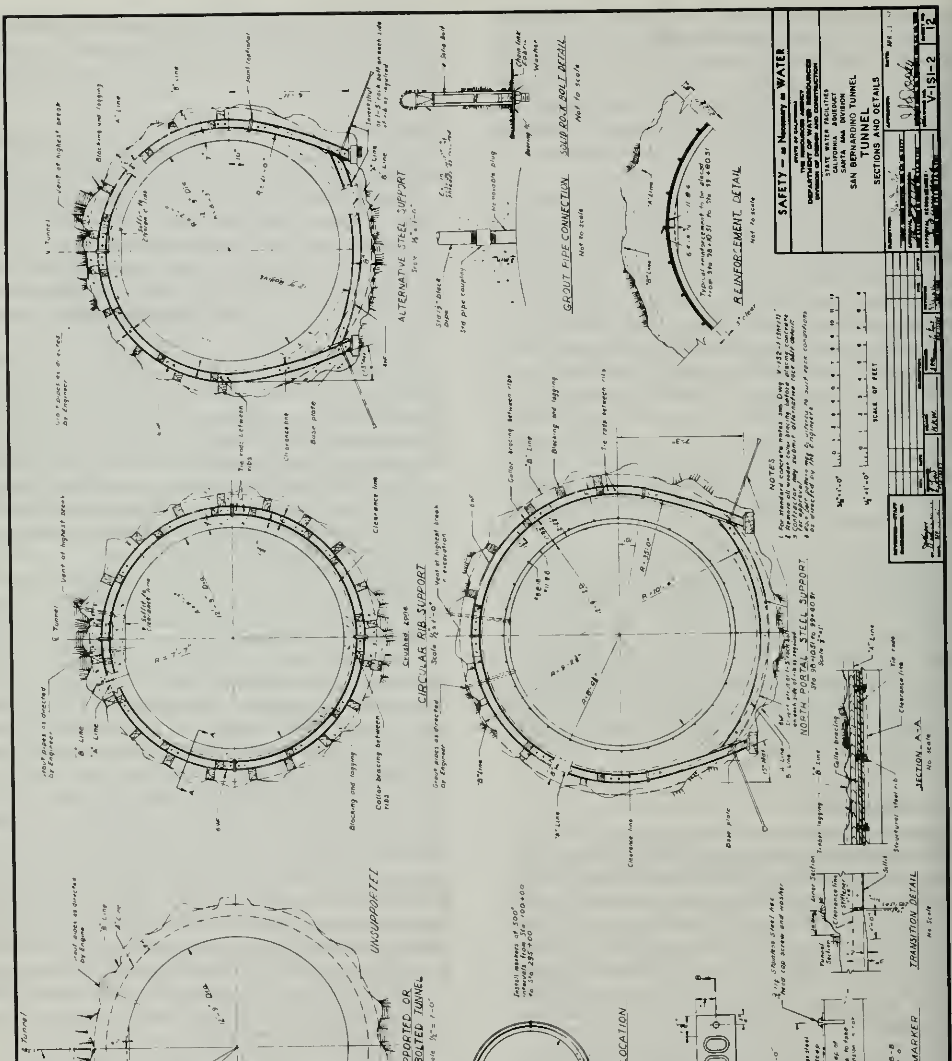


Figure 263. San Bernardino Tunnel—Sections and Details

TABLE 25. Major Contracts—Santa Ana Division

	Specification	Low bid amount	Final contract cost	Total cost-change orders	Starting date	Completion date	Prime contractor
San Bernardino Tunnel Mile 408.1 to Mile 411.5	67-39	$21,205,956	$24,902,072	$709,298	9/ 5/67	2/ 3/72	J. F. Shea Co., Inc. and John F. Shea and Edmond H. Shea, Jr.
Pipeline—Devil Canyon Powerplant to Mill Street Mile 416.0 to Mile 424.5	70-04	10,509,195	10,393,401	32,958	7/ 6/70	5/11/72	Perini Corporation
Pipeline—Mill Street to Sugarloaf Mountain Mile 424.5 to Mile 431.5	70-16	11,390,000	11,922,263	285,761	8/20/70	7/19/72	Zurn Engineers
Pipeline—Day Street to Ellsworth Street Mile 436.6 to Mile 437.3	69-02	838,888	849,136	139	3/15/69	10/17/69	Pascal and Ludwig
Pipeline—Sugarloaf Mountain to Lake Perris Mile 431.5 to Mile 443.0	71-08	13,300,265	13,416,434	119,697	5/29/71	1/31/73	Sully-Miller Contracting Company

Design and Construction by Contract Reaches

Silverwood Lake to Devil Canyon

Design. Alignment and sizing of the San Bernardino Tunnel were determined in accordance with previously described criteria. Geologic conditions were explored extensively for guidance in estimating conditions affecting the tunnel support system and lining thickness.

The original inside diameter was 12.75 feet (Figure 263) to deliver 2,020 cfs. Later, the diameter was enlarged to 13.0 feet by request of the construction contractor because a larger sized tunnel matched his equipment. This change was made at no additional cost.

Steel rib supports were designed by the force polygon method using a design stress of 23,000 psi for A36 steel.

Thickness of the unreinforced-concrete lining was 14.5 inches from the finished surface tunnel to inside of the supports; total minimum thickness from the inside tunnel surface to excavated rock surface was 22.5 inches minimum.

A steel liner was placed at the south tunnel portal where the depth of rock cover was less than 60% of the internal pressure head. This liner was designed to restrain full, internal, hydrostatic and hydrodynamic loads; thus, high-strength, fire-box-type, steel-plate material with a yield stress of 50,000 psi was used. Stiffeners were designed to resist external hydrostatic loads assumed equivalent to the depth of cover in feet of head, with a safety factor of 1.25. Additional working space was provided outside the liner for welding and for placing concrete.

The following factors were considered in selection of north and south portal spoil areas: (1) storage capacity, (2) hauling distance, (3) accessibility to portals, (4) aesthetic considerations and erosion control for spoil pile, (5) accessibility of stockpiled excavated material at north portal for Cedar Springs Dam construction, and (6) future recreation developments near the north portal.

The south tunnel portal was designed to provide a bench 120 feet wide with cut slopes excavated at 1½:1. This large bench was necessary to provide space for the Devil Canyon Powerplant penstock headworks valve, turnout, and maintenance roll-out section. Steel tunnel liner plate was extended 5 feet from the portal face to allow a welded connection with the penstock. A concrete portal structure simplified construction of the Tunnel and protected the portal from rock slides (Figure 264).

Figure 264. South Portal Area—San Bernardino Tunnel

The north portal of the Tunnel was designed to facilitate connection to the intake tower. Also, it provided a channel to carry portal drainage during construction and space to store siltation during operation. Channel side slopes, designed for rapid drawdown and earthquake loading, were cut 3:1 in alluvial overburden, ¾:1 in rock, and 1½:1 in gravel. Benches at 30-foot intervals were shaped and paved as ditches to remove water and ensure optimum slope stability (Figure 265).

The Tunnel was designed to be pressure-grouted, at locations determined by needs identified during construction, through holes drilled around the tunnel lining at 60-degree intervals. Consolidation grouting was applied at pressures up to 200 psi and contact grout to 30 psi. Cutoff grouting was applied to stop ground water inflow.

The San Bernardino intake tower (Figure 266) was designed to release 2,020 cfs into the San Bernardino Tunnel. A multiported design ensured that this flow could be withdrawn from various reservoir depths as required. Inlet ports located in six tiers were spaced vertically on 18-foot centers. At each tier, four butterfly valves were installed to provide symmetrical inflow from the same quadrant points on the tower. Nonmoving trash screens were installed over each port (Figure 267).

Optimum conditions for power generation required that the highest possible water surface be maintained inside the tower. To achieve this economy, a 60-inch butterfly valve (Figure 268) was specified for each port. These valves delivered design flow with a differential head of approximately 5 feet between the reservoir and inside of the tower. All valves were designed with a hydraulic operator system controlled from the operating deck at the top of the tower.

The base of this tower was located on a large block of gneissic granodiorite. For design purposes, the block was assumed infinite in extent and to have a strength of 14,000 psi with a modulus of deformation equal to 5,000,000 psi.

The following design criteria were used for the intake tower and tower bridge:

1. Bridge loading—AASHO, 1961, H-10 loading without impact.
2. Tower roof and operating deck loading—Dead load plus 40 pounds per square foot live load on roof; dead load and H-10 or 20 kip concentrated load on operating deck.
3. Static tower loads—ACI 1963 code and UBC, 1961 and 1964, and AISC Code, sixth edition, where applicable.
4. Lateral loads
   a. Wind loads—much less than earthquake loads and, thus, not computed.
   b. Earthquake loads
      (1) Base moment to be computed from 0.2g static seismic acceleration loading on the tower including contained and virtual mass of water.
      (2) Base bearing pressures determined from earthquake base moment combined with full uplift must not exceed 85% of the ultimate concrete strength (0.85 f'_c); provide peripheral anchor bars to resist 10% of the base moment.
      (3) For the conditions in (1) and (2), maximum concrete stress not to exceed 0.85 f'_c and maximum reinforcing steel stress not to exceed 40,000 psi, minimum yield stress for A15 intermediate-grade steel.
      (4) A first-mode deflection shape to determine moments above the base.
      (5) Tower shaft, reinforcing steel; yield stress to be reached before the concrete reaches 0.85 f'_c.

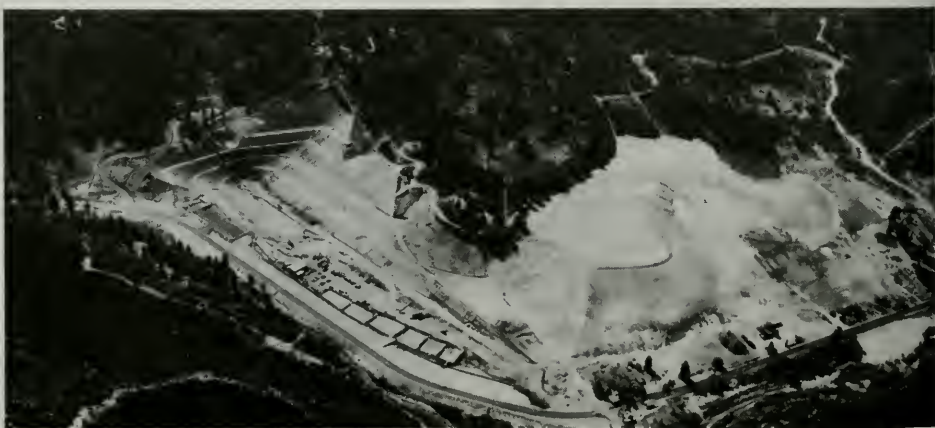


Figure 265. North Partal Area—San Bernardina Tunnel

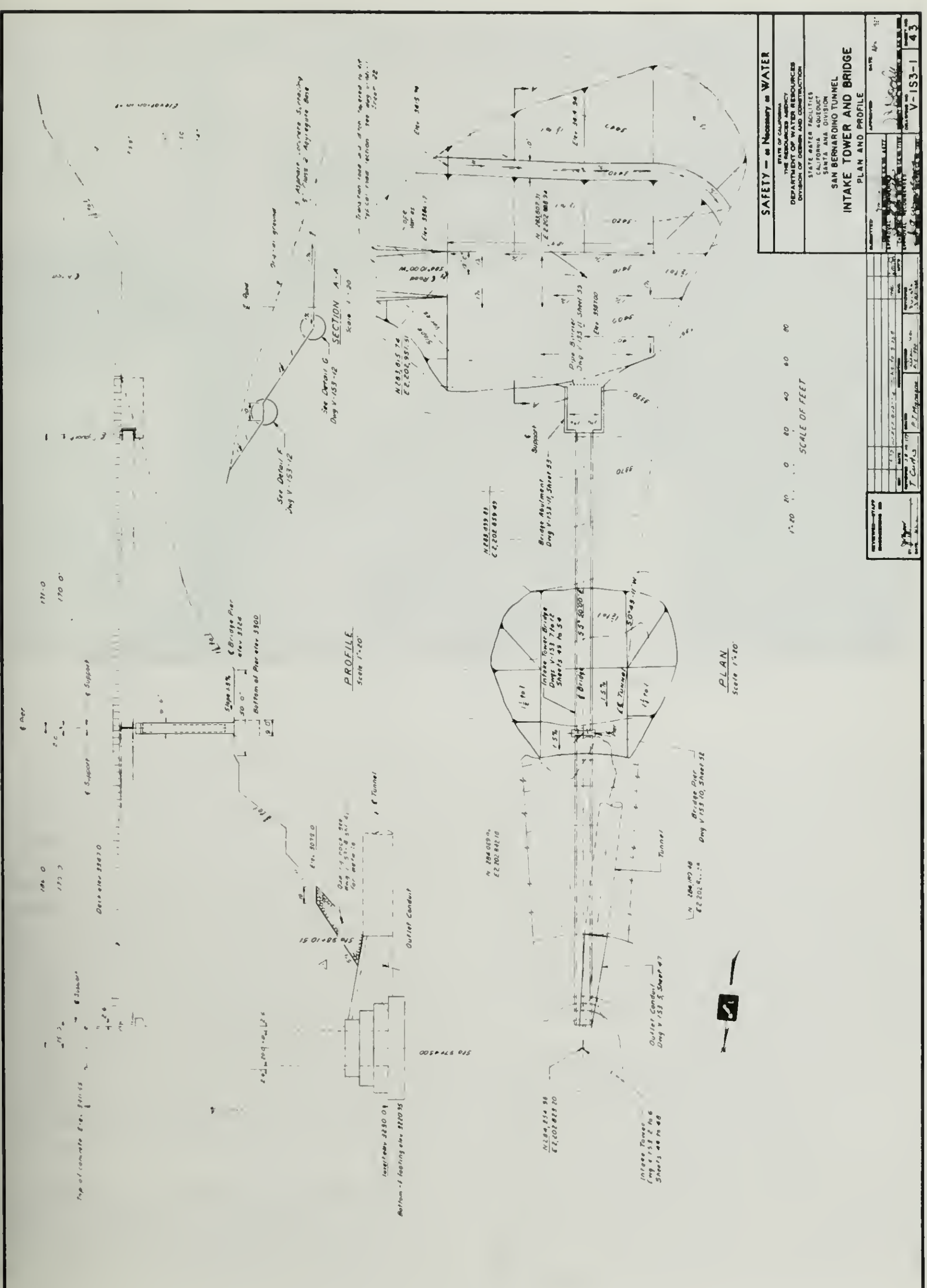


Figure 266. Intake Tower and Bridge—Plan and Profile

Figure 267. Completed North Portal Intake Tower and Bridge—San Bernardino Tunnel

An investigation was made by the Department during the design period on the response of this tower in the plastic region under a loading produced by the hypothetic San Andreas design earthquake. This was 50% greater than the El Centro earthquake of 1940. Predicted effects on the tower were confirmed; yielding of steel reinforcement would occur, but the structure would not be destroyed and would be repairable.

Construction. The Tunnel was driven simultaneously from both north and south portals. The south heading was excavated using a rail-mounted jumbo and mucker while the north heading used rubber-tired equipment (Figure 269).

A pilot hole preceding the tunnel heading was specified in the contract for advance warning of large inflow volumes and high-pressure ground water conditions. This precaution was important because ground water stopped mining operations intermittently at both headings. Grouting was used to control ground water inflow which reached a maximum of 2,000 gallons per minute. At one location, a concrete plug was necessary at the heading to contain grouting pressure necessary to stop the inflow. A Parshall flume was installed at each portal to measure and record the quantity of ground water leaving the Tunnel (Figure 270).

An average of 50 to 55 holes, 4.5 to 5.0 feet deep, were drilled and 120 to 170 pounds of 60% strength dynamite was used for each round in the tunnel excavation cycle.

Steel rib sets generally were placed on 4-foot centers. Maximum spacing was 5 feet and minimum spacing was 2 feet. Steel and wood lagging was used.

When the south tunnel heading had advanced beyond the surge chamber junction, the surge chamber side drift was completed and a $7\frac{5}{8}$-inch-diamete pilot hole was drilled vertically downward from th surface to the side drift on the surge chamber center line. Hoist cable was threaded through the hole t support a platform from which a 7-foot-diameter pilo shaft was raised. This shaft was extended upward fo 206 feet until the rock condition became unstable, an the remaining 64 feet was mined downward from th surface.

Enlargement of the surge chamber excavation to th completed 45-foot diameter proceeded by work cycl of drilling, blasting, and mucking with a backho Rock material was allowed to fall through the 7-fo shaft and hauled out through the south portal. Durin

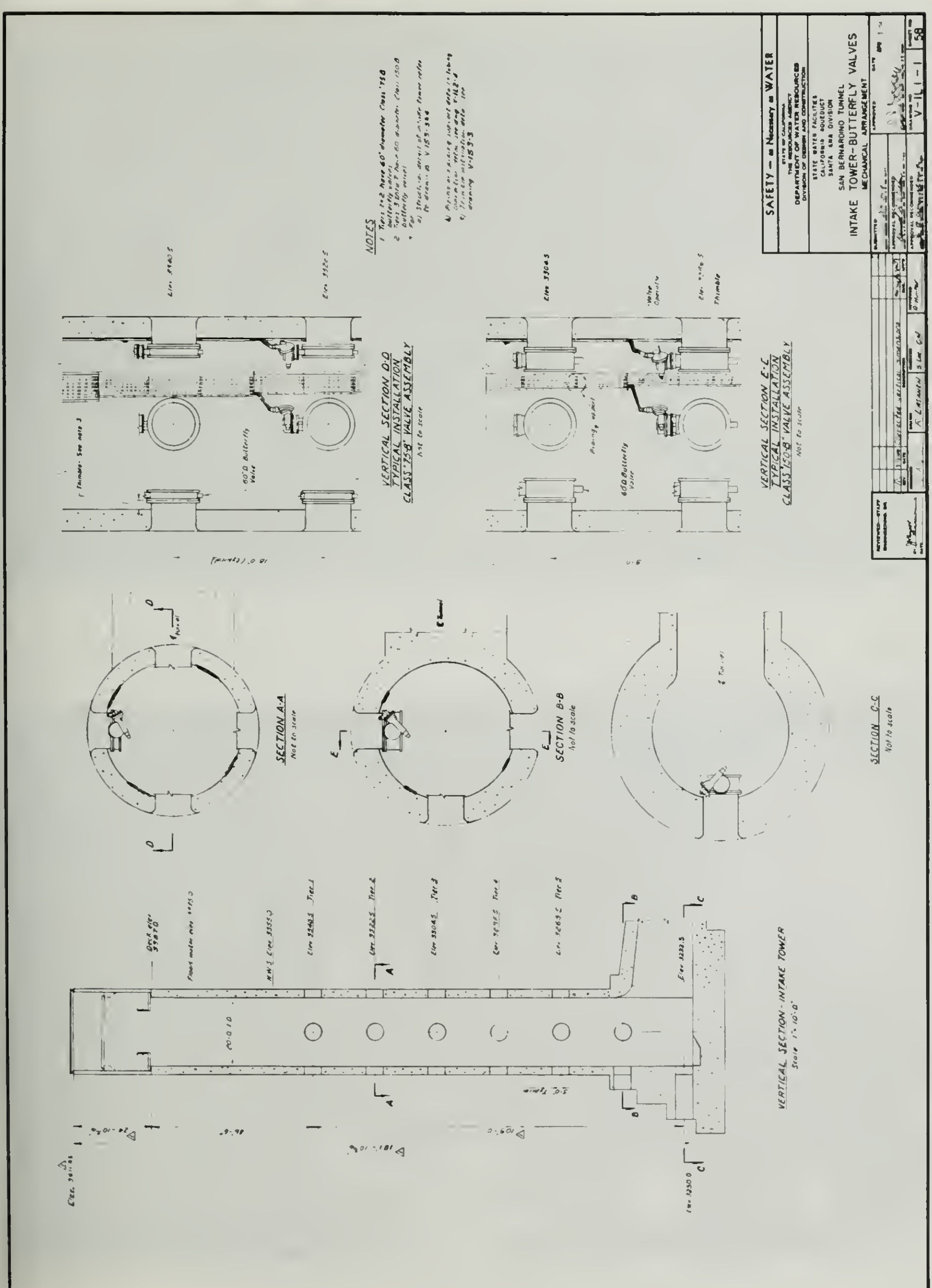


Figure 268. Intake Tower Butterfly Valves—Mechanical Arrangement

Figure 269. Installing Track at South End of San Bernardino Tunnel

Figure 270. Sandbag Bulkhead Used to Control Water in San Bernardino Tunnel

this work, the chamber excavation was gunited to protect miners from loose rocks (Figure 271).

When the main tunnel was holed through, rails from the south heading were extended throughout the Tunnel to carry lining forms and concrete placing equipment. Unreinforced-concrete lining then was placed on a continuous basis, six days a week, proceeding from north portal to south portal.

Concrete lining for the surge chamber was placed continuously using timber slip forms including the 55-foot aboveground portion.

For the intake tower footings, concrete was placed in three consecutive, 9-foot-high, stepped lifts at approximately one-month intervals. Concrete for the tower was placed with conveyor belts and a concrete pump up to the height limit of the pump. Remaining lifts were placed with a crane using 1-cubic-yard buckets. The intake tower bridge girders were erected directly on the concrete tower, pier, and abutment.

Day Street to Ellsworth Street

Design. This short reach was the first constructed on the Santa Ana Valley Pipeline. Design and specification concepts developed from experiences on this contract were utilized for other reaches (Figure 272).

This portion of the Pipeline consisted of 3,500 feet of 10-foot-diameter, prestressed-concrete, cylinder pipe located within the Sugarloaf Mountain to Lake Perris reach of the Pipeline. Because it passed under Riverside Raceway, it was scheduled to be constructed during an idle season at the automobile raceway (Figure 273).

Specifications provided for bidding alternatives using either prestressed-concrete cylinder pipe (PCCP), reinforced-concrete cylinder pipe, or ring-stiffened steel pipe. Bids were received only on the PCCP alternative (Figure 274).

Blowoffs and turnouts were not required in this reach, but two combination air release-vacuum valves were located at high points in the Pipeline.

Construction. Major portions of the trench excavation were made with two 14-cubic-yard scrapers and tractors equipped with rippers. One tractor also was equipped with a slopeboard for shaping the sides of the pipe trench.

Although a trench was specified to be 8 feet wide at the bottom, the contractor elected to widen the trench excavation to accommodate the scrapers. Otherwise, it would have been necessary to excavate the bottom 5 or 6 feet of the trench with a dragline or backhoe. By using wheeled excavators, the bottom trench was cut between 14 and 20 feet wide, which expedited excavation but greatly increased the amount of backfill required (Figure 275).

Final grading of the trench bottom was made with a tractor with ripper, a loader, a grader, and a self-loading scraper.

Prestressed-concrete pipe was manufactured at Corona, California. The 10-foot-inside-diameter pipes were made in 16-foot sections, hauled to the site on low-bed trailers, and unloaded by a 150-ton crawler crane. Pipe sections were placed directly into the trench or unloaded nearby for temporary storage (Figure 276).

Valuable experience was gained from attaching the corrosion protection system for the concrete pipe (Figures 277 and 278). This was specified to include direct bonding of electrical connections to the prestressing wires on the pipe. Heat, generated during a silver soldering process used to make bonded connections from the steel cylinder to the prestressing wires, caused some of the highly tensioned wires to fail. Twenty-eight sections of pipe were found to have damaged prestressing wires. Since the average longitudinal extent of defective prestressing wires was less than 4 inches per location, repairs were made by welding an 8-inch-wide, $\frac{5}{16}$-inch-thick, steel band around the pipe at each location with defective wires. The steel band then was covered with a coat of gunite.

Sand for consolidated backfill was hauled from Santa Ana River sources. Two 6-inch and two 4-inch vibrators and two water jets were used to consolidate the backfill material. The wide overexcavation of the trench bottom made it difficult to achieve the required density. Consolidated sand backfill extended to 3 feet above the bottom of the trench. Backfill above that level was made with excavated soil material (Figure 279).

Specifications required the Pipeline to be tested to 100 psi for 24 hours, with total leakage not to exceed 2,000 gallons. High-pressure testing of an isolated section of pipeline such as this would require unusual risks because of the limited longitudinal strength in prestressed, concrete pipe. Testing procedures were revised to allow an initial test at 30 psi. This short pipeline reach remained full of water and then was tested at 100 psi along with the adjoining pipeline when the latter was completed.

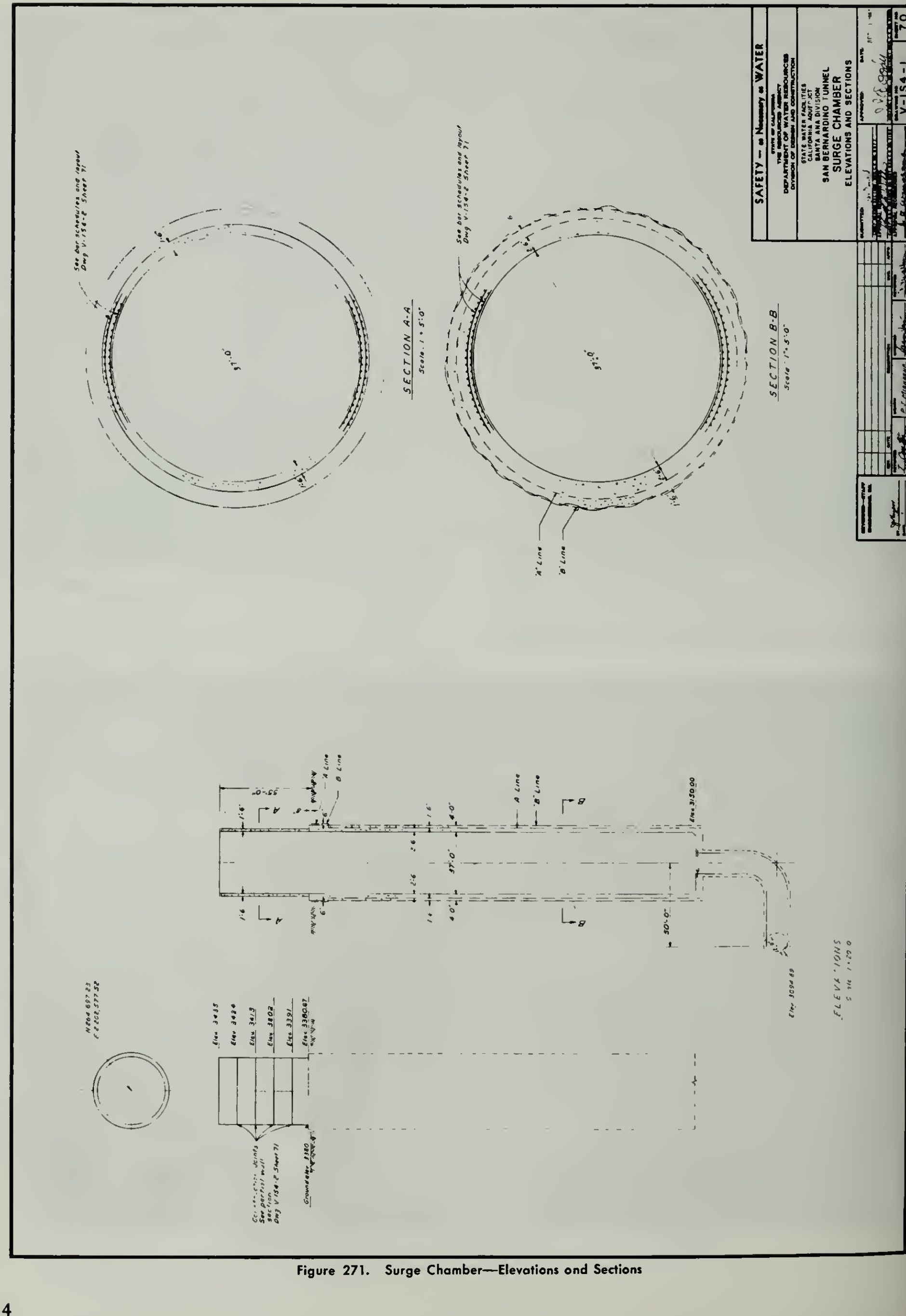


Figure 271. Surge Chamber—Elevations and Sections

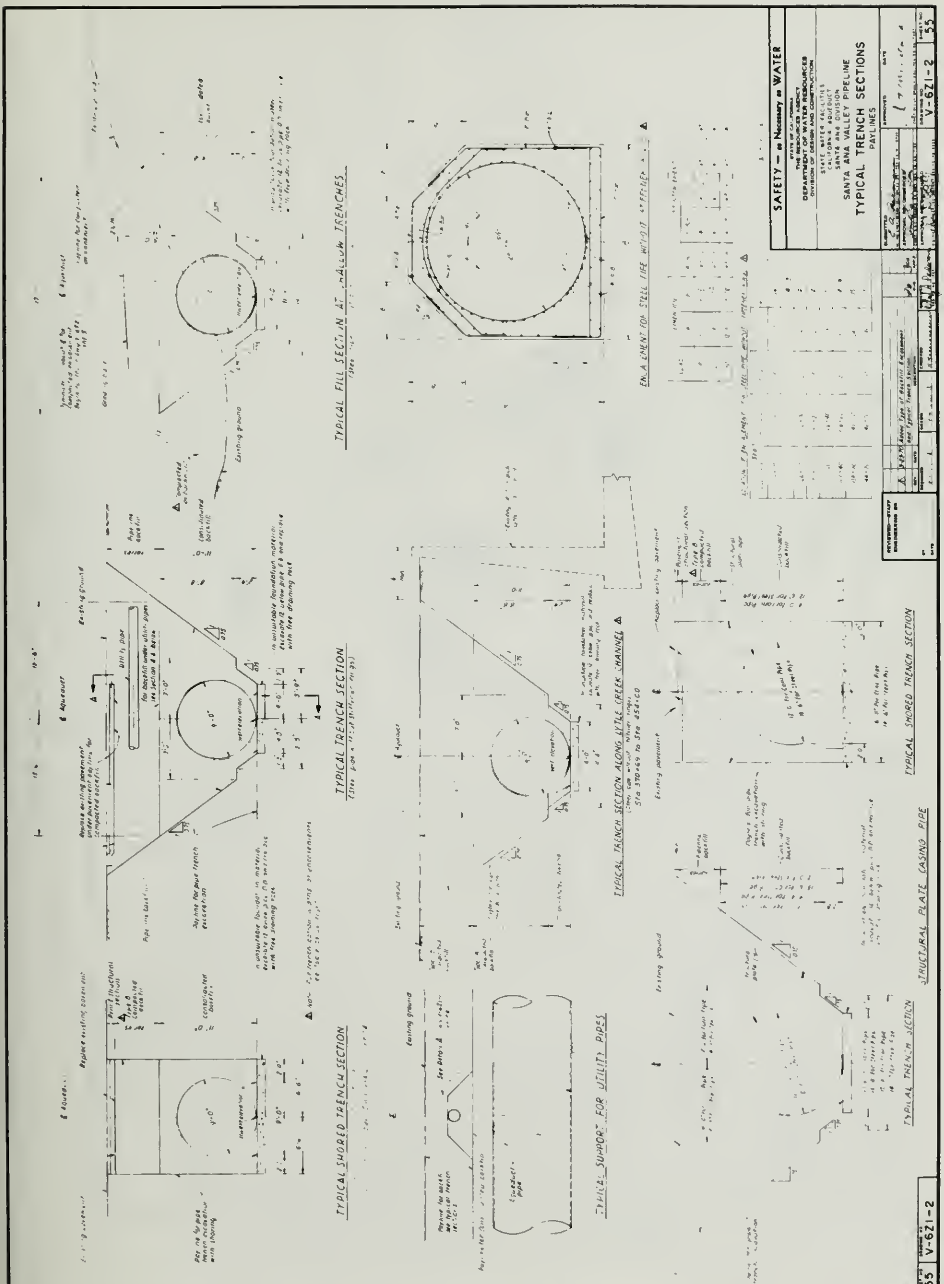


Figure 272. Typical Trench Sectians

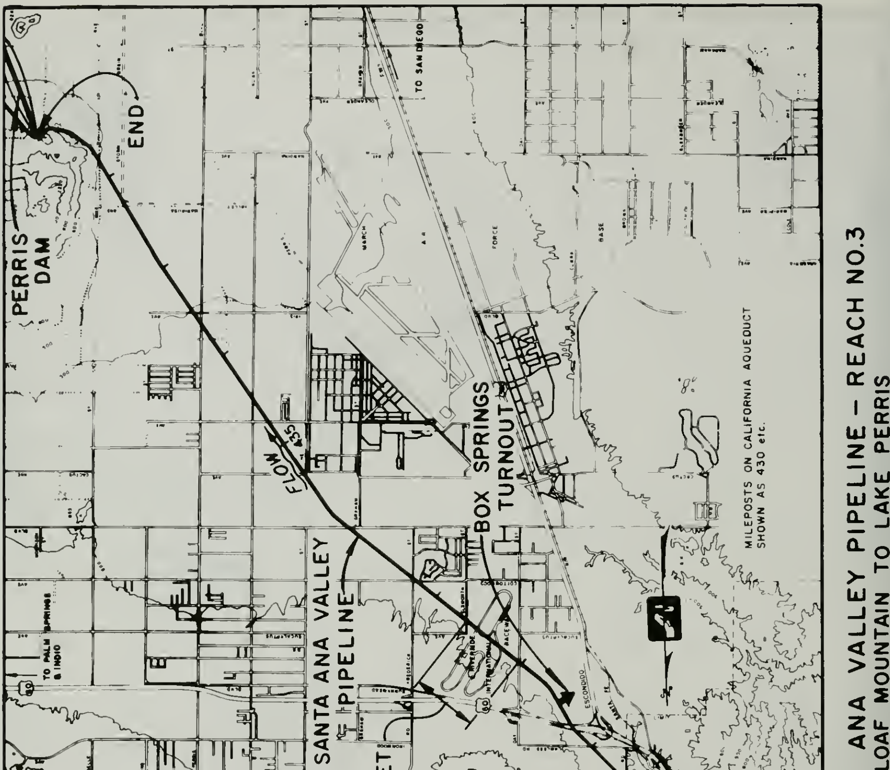


Figure 273. Santa Ana Valley Pipeline Plan—Riverside Raceway

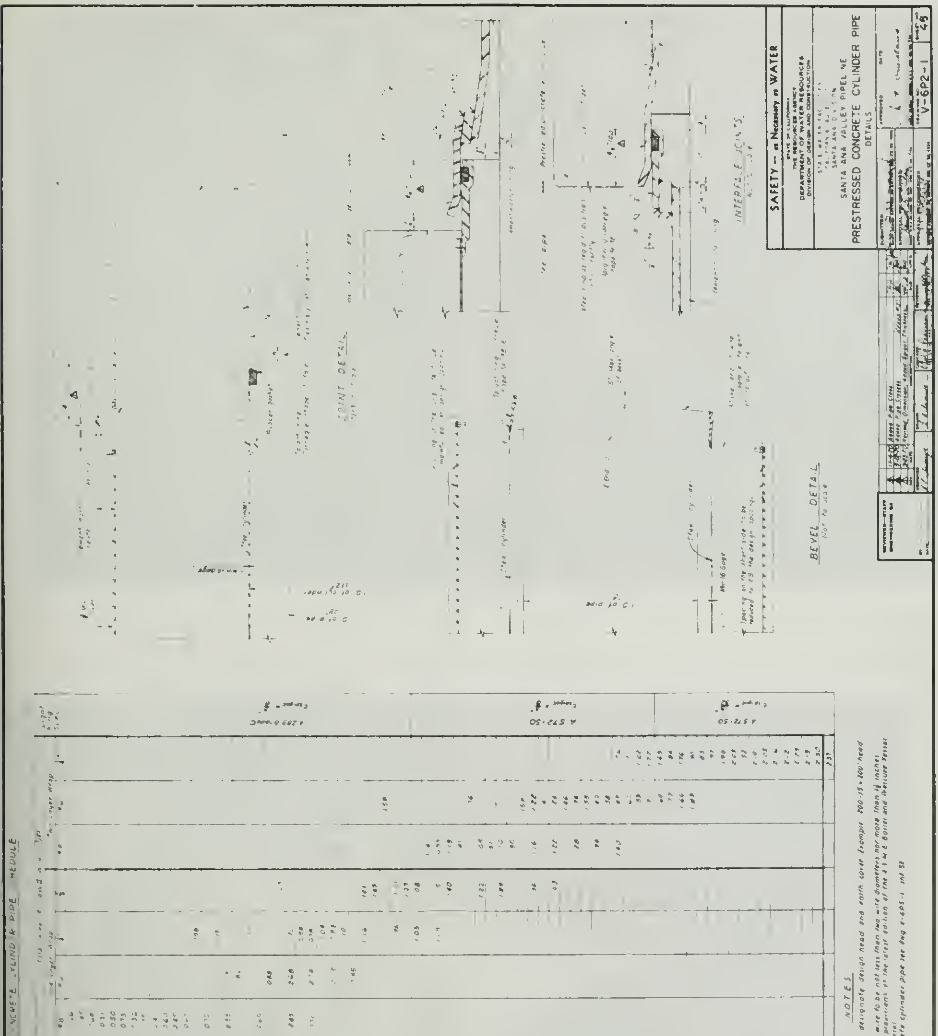


Figure 274. Prestressed-Concrete Cylinder Pipe Details

Figure 275. Pipe Trench at Riverside Raceway

Figure 276. Setting Pipe Sectian in Trench

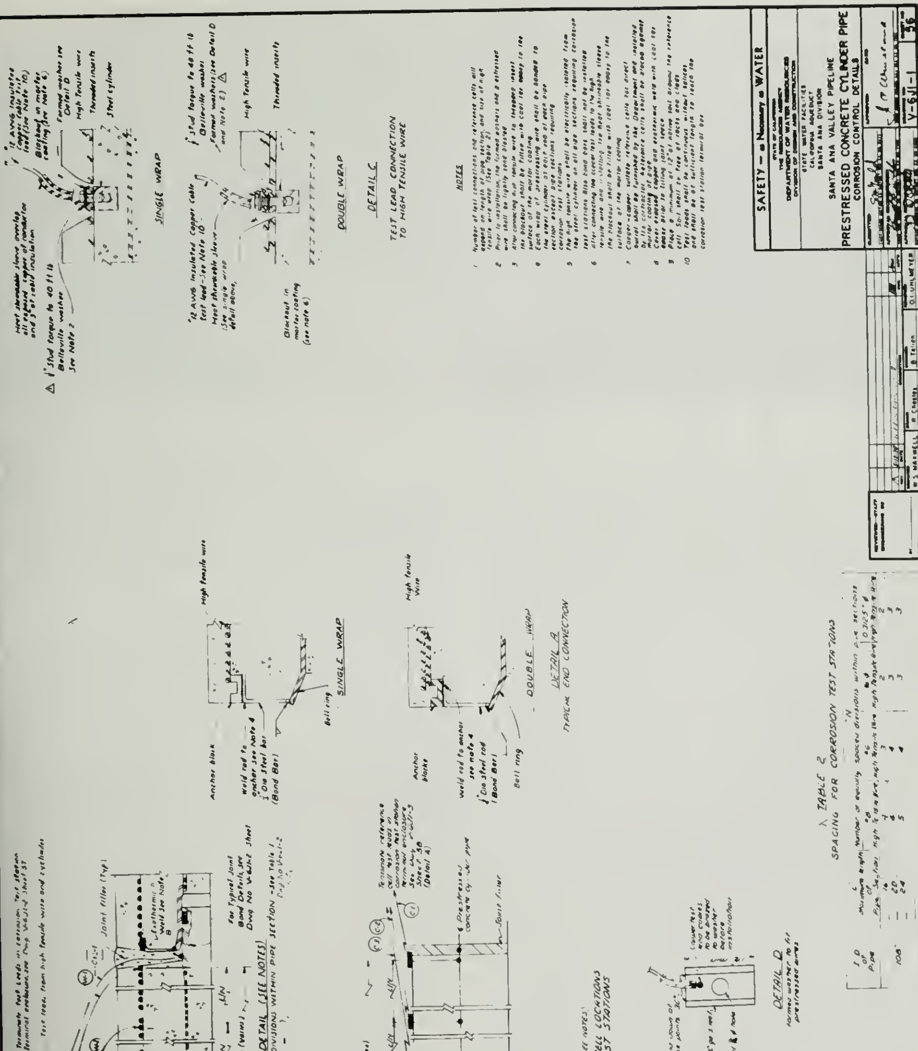


Figure 277. Prestressed-Concrete Cylinder Pipe—Carrosion Control Details

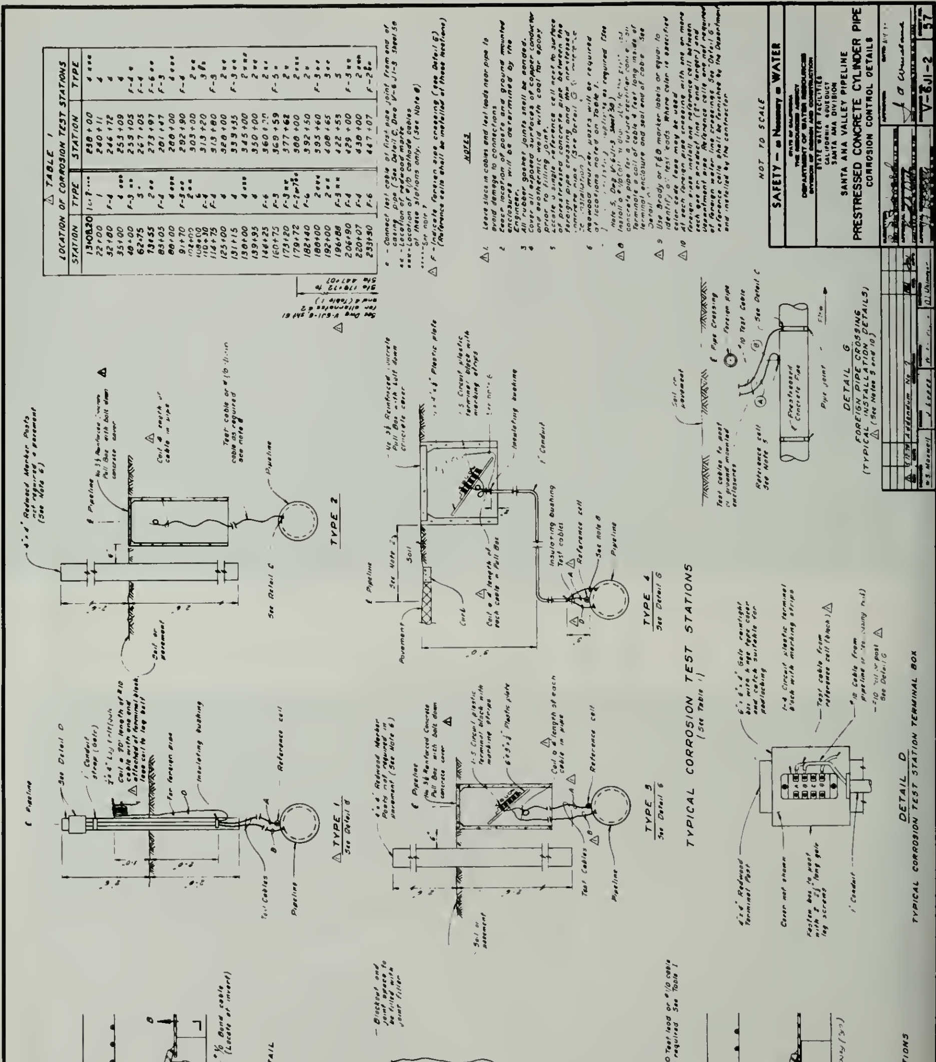


Figure 278. Prestressed-Concrete Cylinder Pipe—Corrosion Control Detoils (Continued)

Figure 279. Placing Pipe Backfill

Devil Canyon Powerplant to Mill Street

Design. This 8.3-mile-long reach of the Santa Ana Valley Pipeline was designed using typical, 9-foot-diameter, prestressed-concrete, cylinder pipe. Design pressure heads on the pipe ranged from 125 to 850 feet (Figure 280).

The contract specifications were advertised with five bidding alternatives which included PCCP, stiffened steel pipe, unstiffened steel pipe, and combinations of PCCP with stiffened and unstiffened steel pipe. Bids were received only on the PCCP alternative.

This reach of pipeline required only one turnout, located near Rialto Avenue at Station 396+50 (Figure 262). This structure was designed to release 12.5 cfs to the San Bernardino Valley Municipal Water District through a 24-inch ball valve (Figure 281).

Approximately 2 miles of this pipeline reach was routed along State Street in the unincorporated areas northwest of the City of San Bernardino. Many homes located along this street made large pipeline construction exceptionally difficult. Pipe trenches were designed with vertical sides and excavation was specified to be removed to stockpiles during construction. Only 500 feet of trench was opened at one time, and all household utilities were maintained in service.

All main highway and railroad crossings were designed as mined tunnels supported by casing pipe through which line pipe was installed.

Construction. After shored trench sections were excavated, corrugated-metal sheeting was used for support with walers held in place by hydraulically controlled bracing (Figure 282).

In the areas where shoring was not required, backhoes were used to excavate the trench. Fine grading in all trenches was done with a small tractor-dozer.

In most cases, pipe was laid by lifting it directly off truck trailers and placing it directly in the trench. Trench-side storage space for pipe sections usually was unnecessary and the crane handled the pipe only once at the site. Two cranes were used for pipelaying at two locations. Only one laying crew was needed because pipelaying was alternated with backfilling at the two locations (Figure 261).

Consolidated pipe backfill was specified to be 2 feet-8 inches deep to give a 120-degree bedding angle for the pipe. A sand slurry was used to place this type of backfill up to the top of the pipe. Required consolidation was achieved using large vibrators. When the slurry had been placed and vibrated over the top of the pipe, the remainder of the trench was backfilled with excavated material (Figure 283).

Pipe in this contract was laid on a continuous downward slope from Devil Canyon to Mill Street, and air release-vacuum valves were not required.

All excavation for casing pipe (Figure 284) at highway and railroad crossings was done with hand-held air tools. Liner-plate casing was installed close behind the excavation heading and a small loader was used for mucking out the tunnel.

At the Interstate Highway 15 crossing, pipe was installed under a bridge (Figure 285). This contract reach required the pipe to be hydrostatically tested in place by bulkheading the pipe ends and filling the pipe with water to elevation 1,938 feet (Figure 286). The lowest point on this pipeline reach also was at the junction with the adjoining downstream contract reach of pipeline. Static pressure head at the junction during test was 904 feet for this reach and 839 feet for the other. By making the two tests simultaneously, a substantial saving in costs for test anchorages was realized because the differential head on the common bulkhead was reduced to only 65 feet of water. The upper bulkhead in this reach was loaded with 95 feet of water and easily anchored to existing facilities.

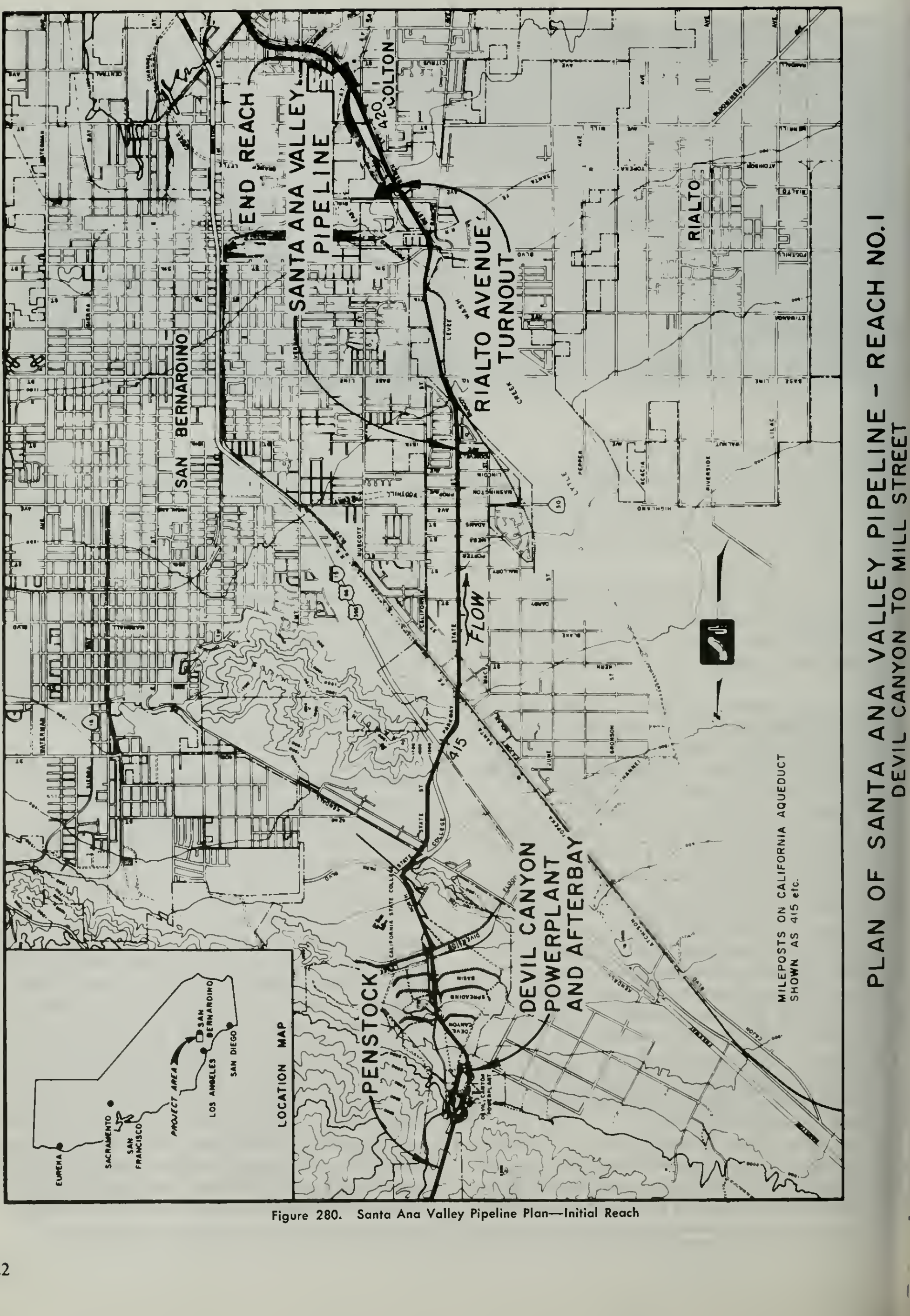


Figure 280. Santa Ana Valley Pipeline Plan—Initial Reach

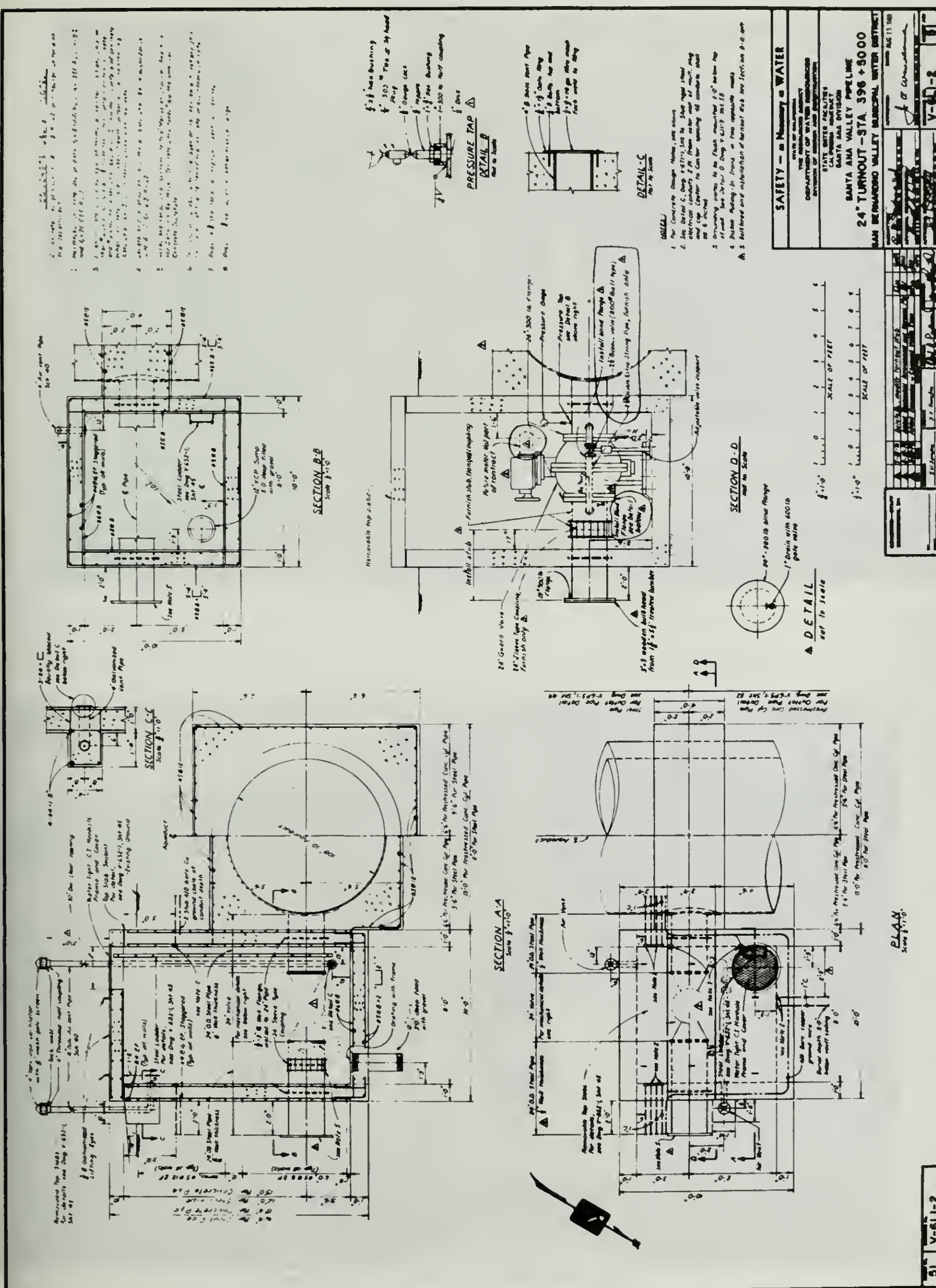


Figure 281. 24-Inch Turnout

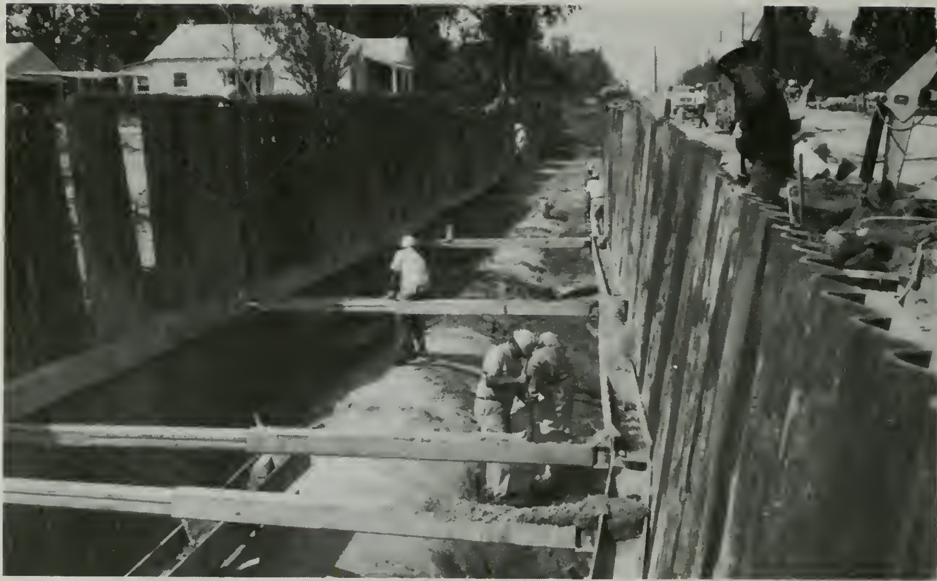


Figure 282. Backfilling Pipeline Along State Street

Figure 283. Vibrators Consolidating Sand Slurry Backfill

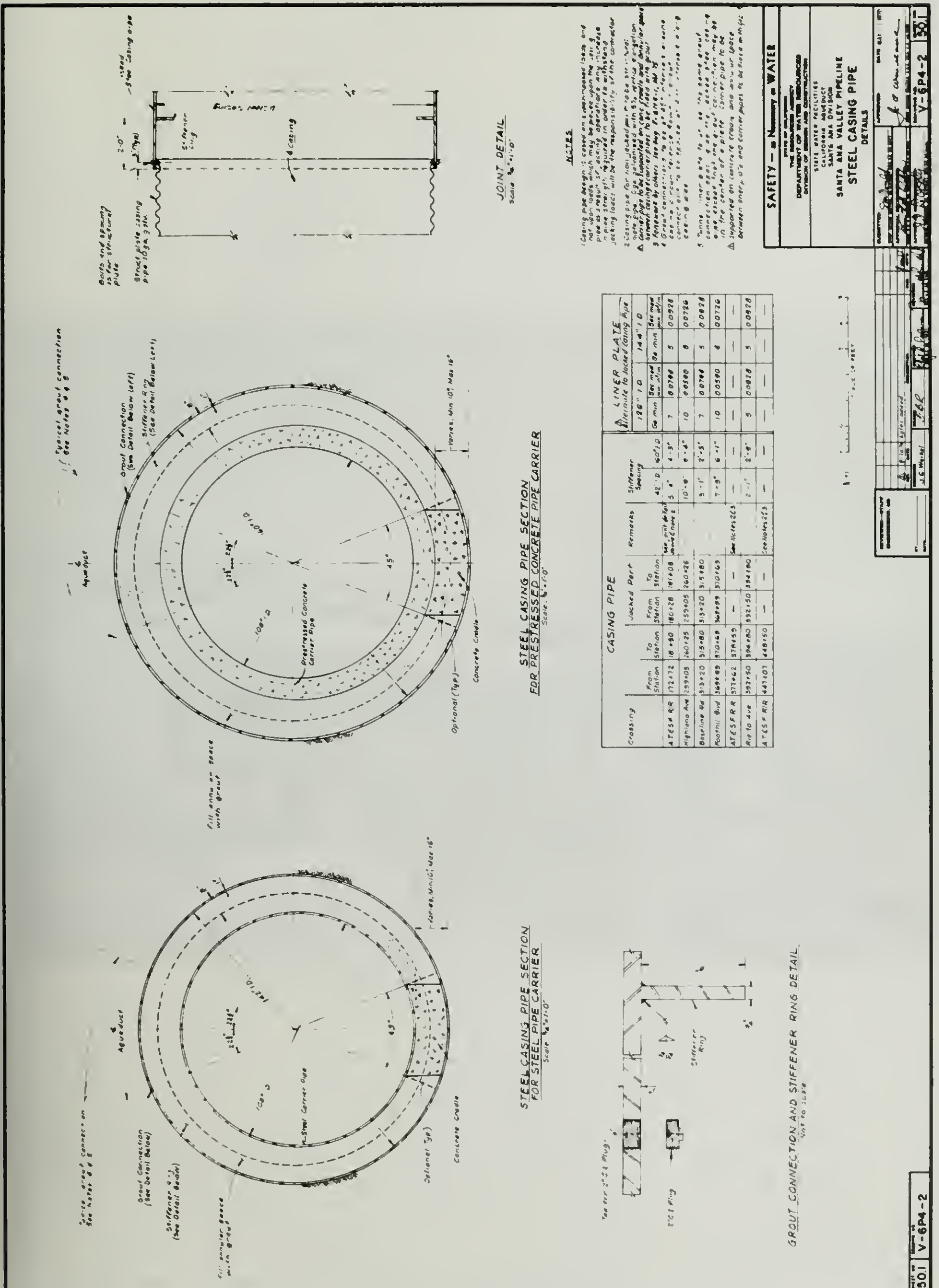


Figure 284. Steel Casing Pipe Details

Figure 285. Pipeline Crossing Under Interstate Highway 15

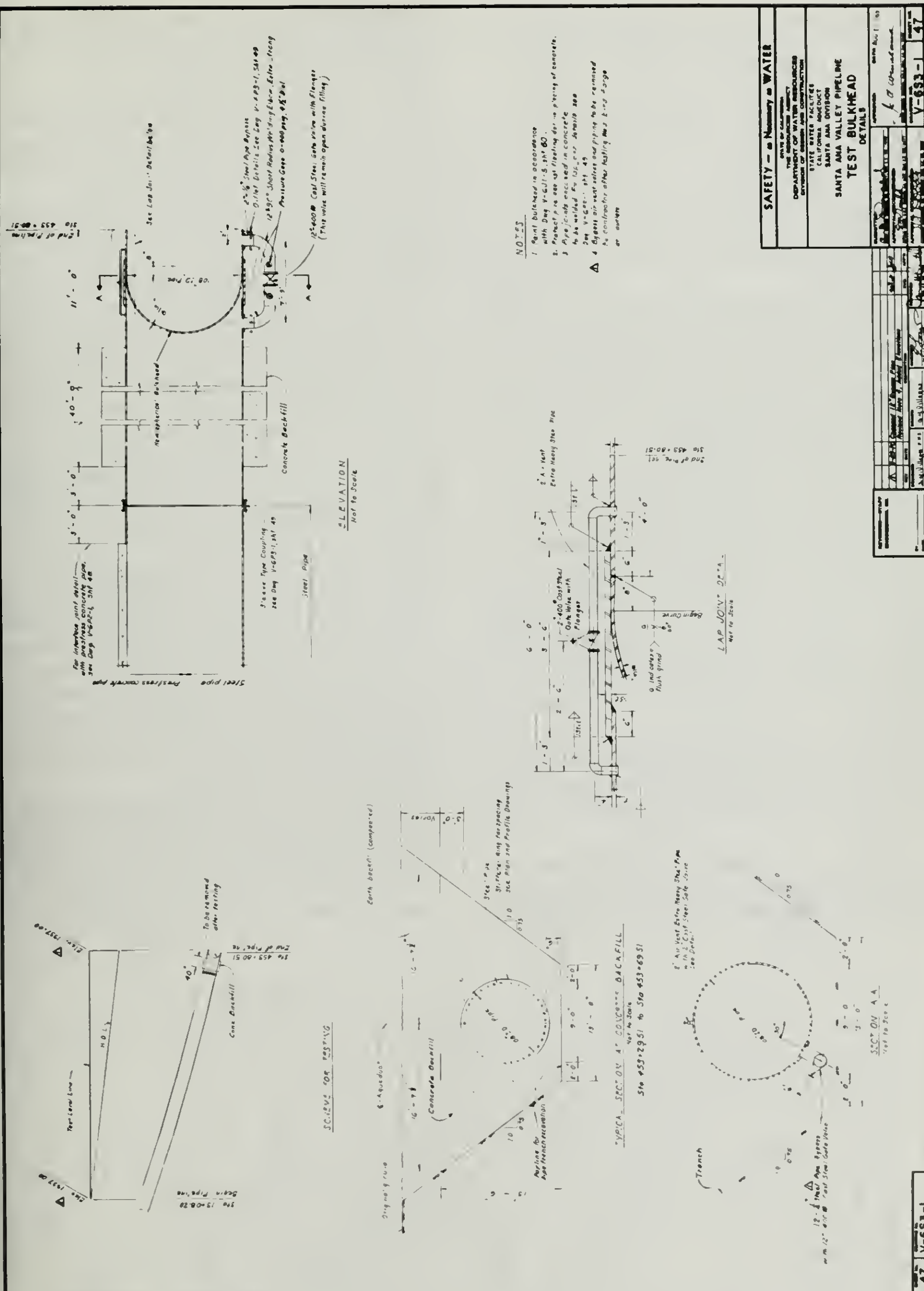


Figure 286. Test Bulkhead Details

Mill Street to Sugarloaf Mountain

Design. This reach of the Santa Ana Valley Pipeline is 7 miles long and was constructed entirely of 9-foot-diameter PCCP (Figure 287).

Contract specifications typically contained alternative bidding schedules for PCCP, stiffened steel pipe, and unstiffened steel and, as in the first two pipeline contracts, bids were received only on PCCP (Figure 288).

This reach required 11 combination air release-vacuum valves, three blowoffs, and two turnouts. Designs were typical with the previous portions of the Pipeline.

The blowoffs at Stations 151+50 and 430+60 (Figures 262 and 289) use submerged sleeve valves to dissipate high head energy. Flow through these valves is regulated by movement of one outside section of pipe over an inner section. The inner pipe was perforated and movement of the outer pipe, or sleeve, governs the number of perforations exposed. These valves were mounted vertically in rectangular concrete structures.

Design head on the valve at Station 151+50 is 860 feet and at Station 430+60 is 780 feet. This first valve operates at a head differential of 500 feet and the second at 450 feet. Flow from these valves discharges over weirs located near the tops of the structures.

All blowoffs and turnouts include guard valves located close to the main pipeline. This allows discharge valves to be serviced and affords added protection against leakage from a single valve. Both guard valves for the blowoff at Station 226+36 and the turnout at Station 226+44 are lodged in a single large vault (Figure 262).

Flow from the blowoff at Station 226+36 is dissipated by means of a fixed-cone dispersion valve. This valve sustains 920 feet of water pressure when closed and can dissipate this full head differential in operation.

Construction. This reach starts at Mill Street near the north city limits of Colton and ends at the top of Sugarloaf Mountain, near the north city limits of Riverside. Typical work consisted of constructing the Santa Ana Valley Pipeline and all appurtenant blowoffs, turnouts, combination air release-vaccum valves, and cathodic protection.

Restrictions in the specifications for this reach influenced schedules and methods of construction. Work was concentrated on the Santa Ana River crossing and the tunnels required at railroad and highway crossings (Figures 290 and 291).

Pipeline excavation followed along Lytle Creek in Colton. The narrow right of way and lack of stockpile space required that only short lengths of trench be exposed. Trenches (Figure 292) were excavated with a crane fitted with a dragline bucket. When sufficient trench length was completed, the same crane was used to place a pipe section. Where the alignment was in open country or in orange groves, scrapers excavated the trench. A large backhoe was used to dig sections which otherwise could not be excavated. This occurred near Grand Avenue in Colton where drilling and blasting were required (Figure 293).

Fine grading of the trench and pipelaying procedures were identical to those used for the Devil Canyon to Mill Street reach (Figures 294 and 295). Consolidated backfill material was imported from the bed of the Santa Ana River.

The last 1,000 feet of this pipeline reach was installed on the north slope of Sugarloaf Mountain. In this distance, the designed pipe profile rose 400 feet, much of it on steeper than 3:1 slopes. A concrete cradle with two embedded steel rails was constructed at pipeline subgrade on this hillside. Pipe was hauled to the top of Sugarloaf Mountain from the south side and placed by sliding it down the cradle with a cable controlled by a winch on a tractor anchored at the top of the cradle (Figure 296).

The pipe placed on this contract was manufactured at Etiwanda, California, and used the same design for PCCP as for the two previous contracts. This reach was satisfactorily tested for leakage in conjunction with the Devil Canyon to Mill Street reach, as previously described.

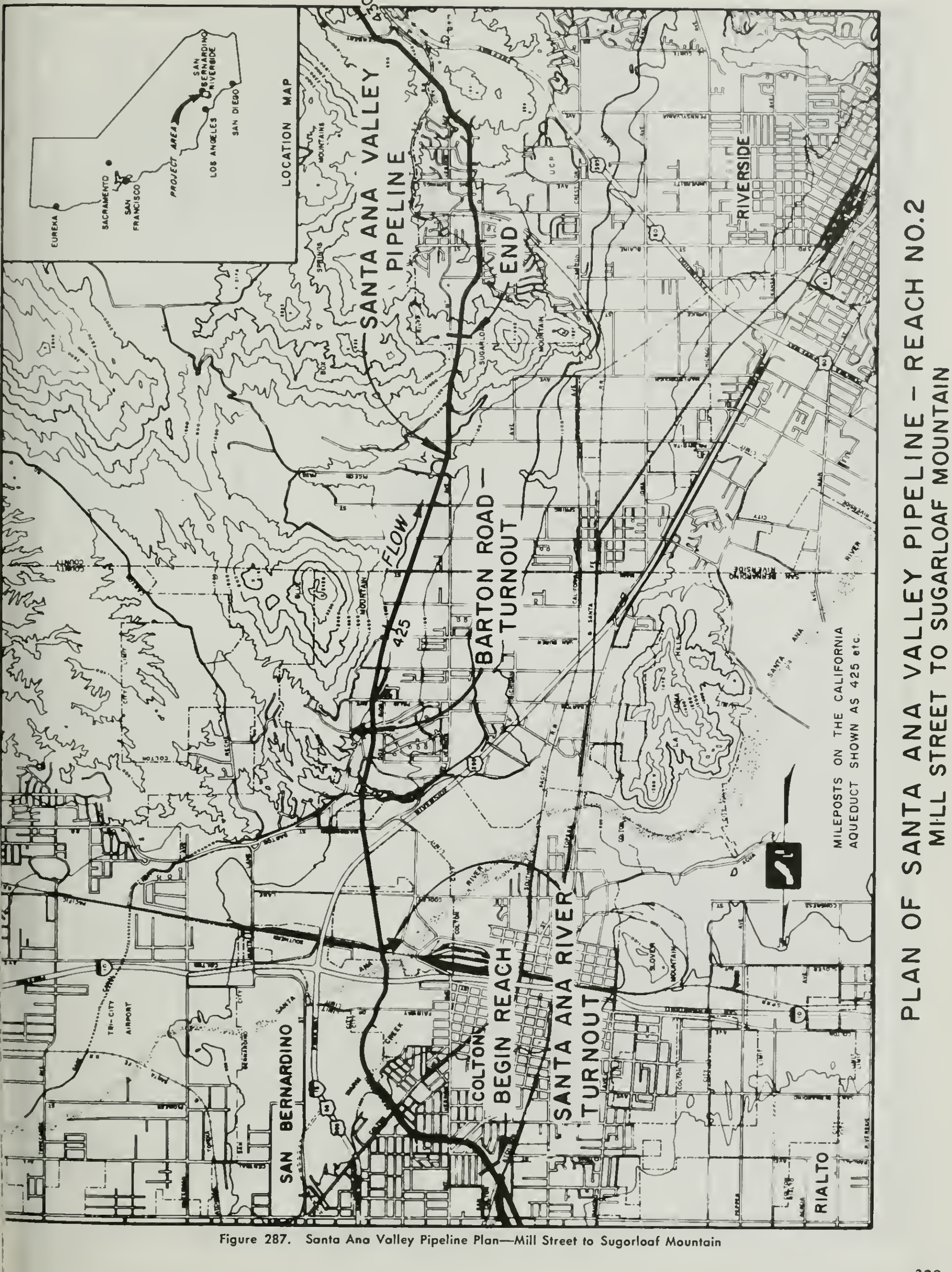


Figure 287. Santa Ana Valley Pipeline Plan—Mill Street to Sugarloaf Mountain

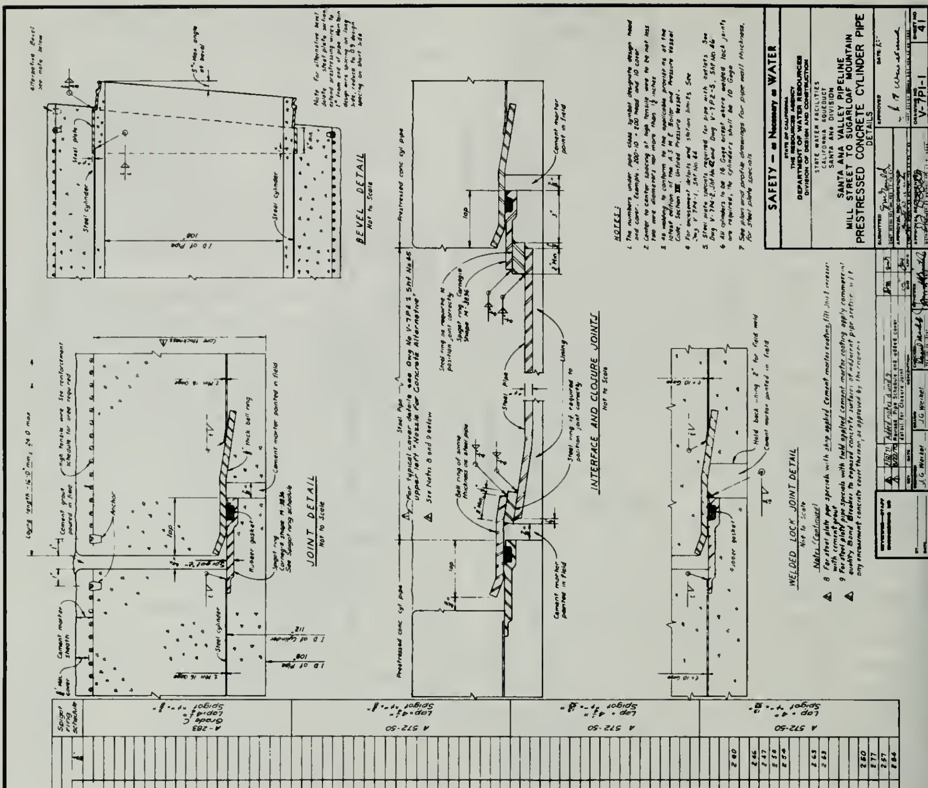


Figure 288. Prestressed-Concrete Cylinder Pipe Details—Mill Street to Sugarloaf Mountain

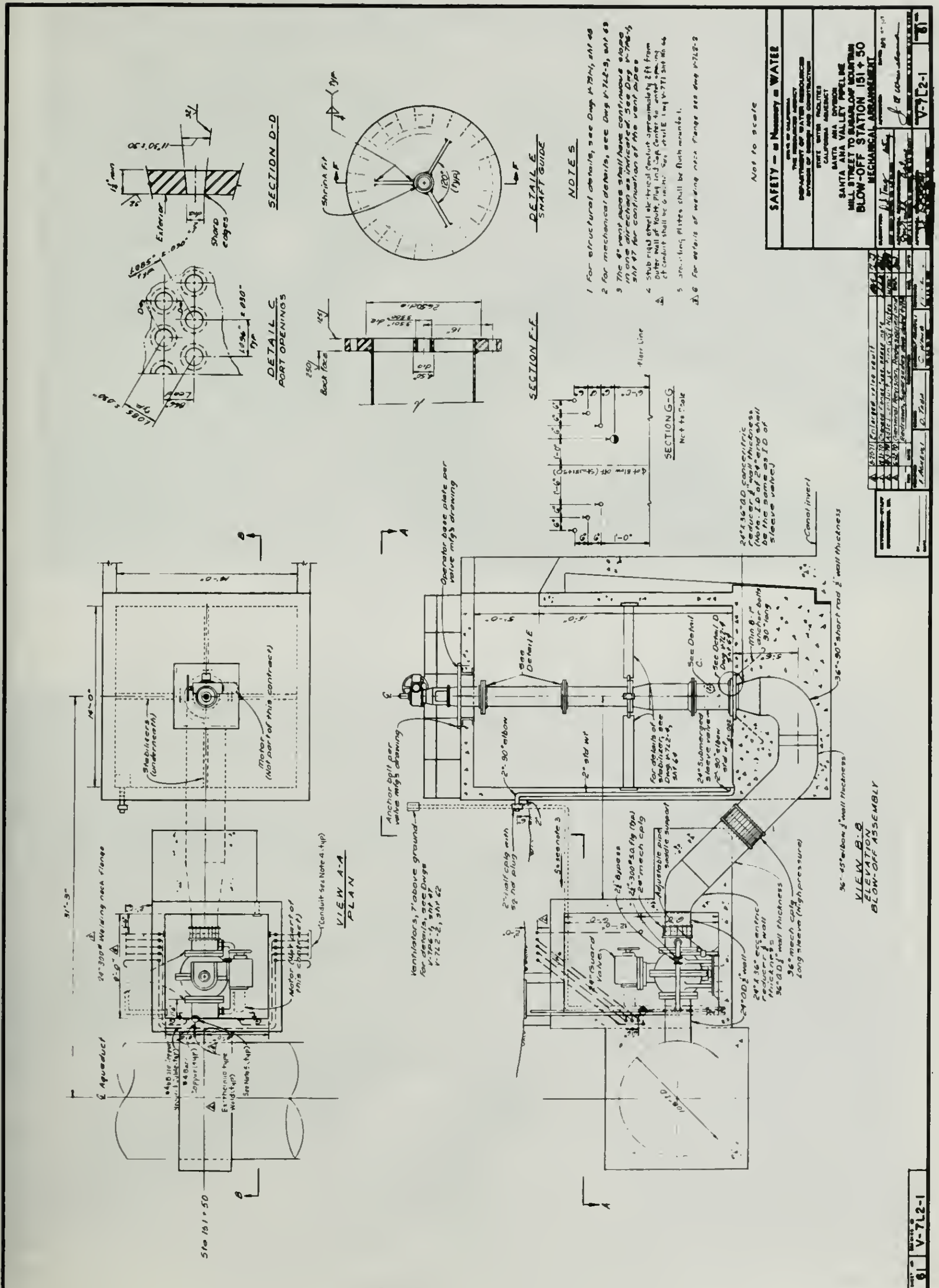


Figure 289. Blowoff Stations—Mechanical Arrangement

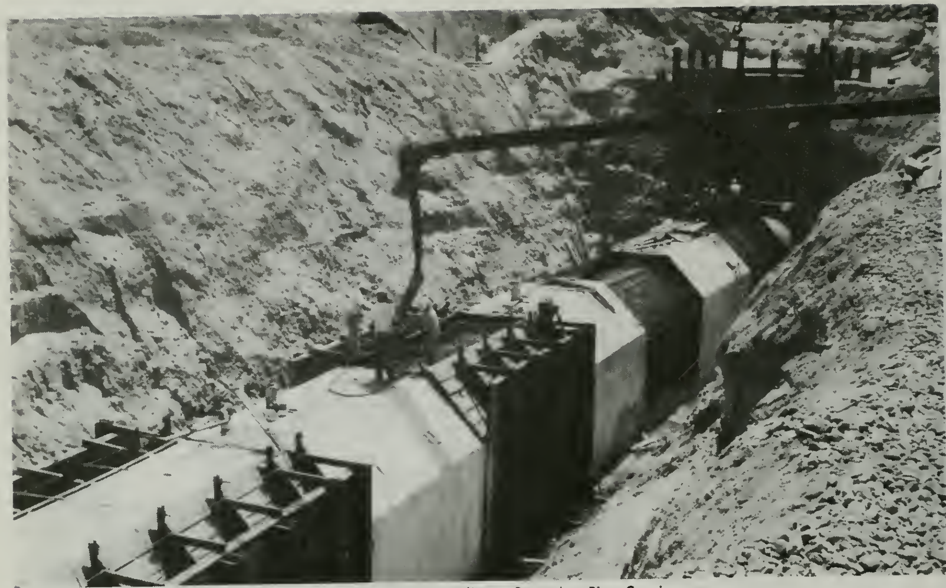


Figure 290. Encasing Pipeline at Santa Ana River Crossing

Figure 291. Pipeline Crossing—Southern Pacific Railroad Main Line

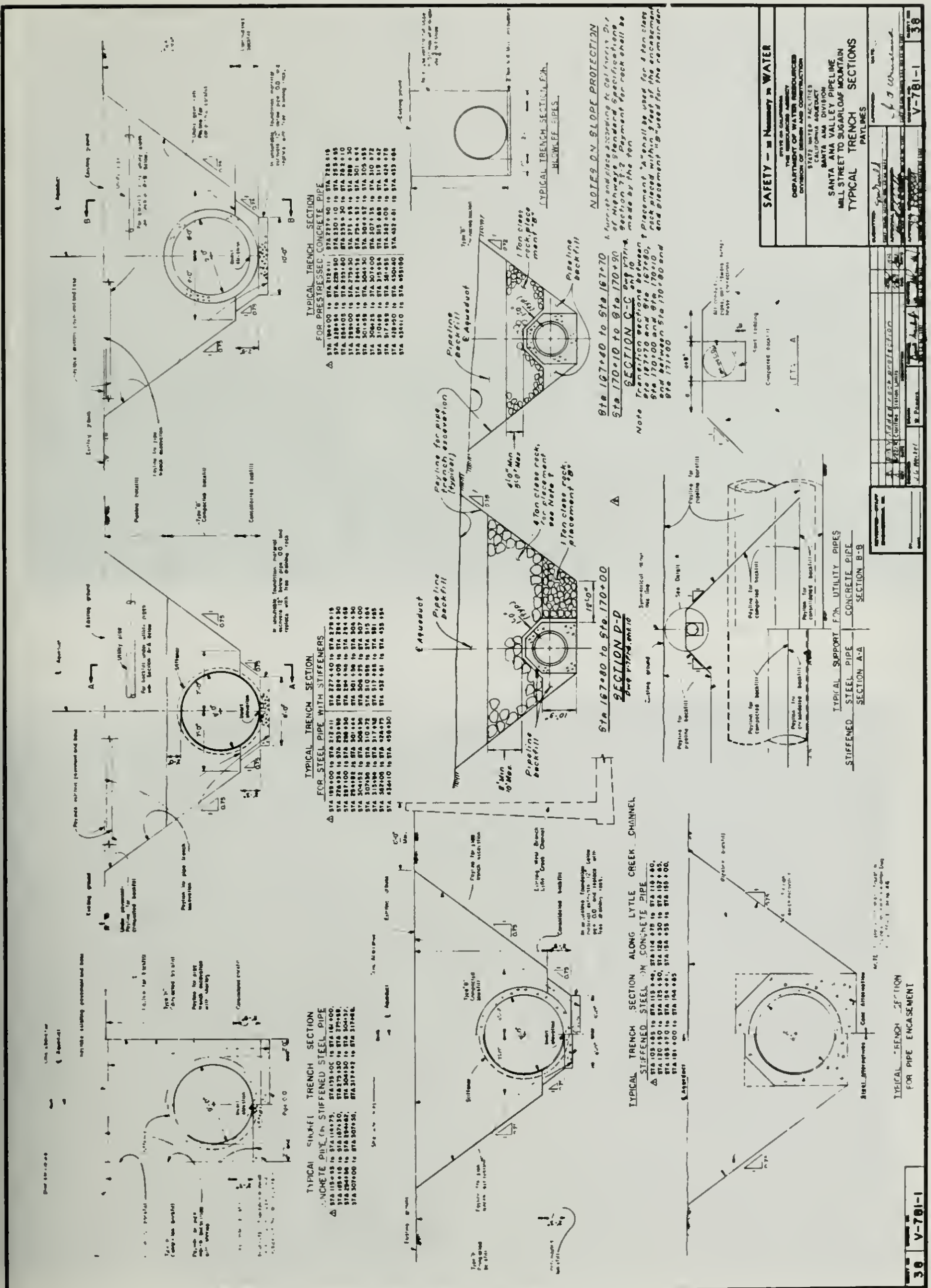


Figure 292. Typical Trench Sections—Mill Street to Sugarloaf Mountain

Figure 293. Pipeloying Along Lytle Creek Flood Control Chonnel

Figure 294. Pipeline Construction Neor Sugarloaf Mountain

Figure 295. Backfilling Pipeline Near Sugarlaaf Mountain

Figure 296. Cancrete Pipe Cradle With Rail Skidway

Sugarloaf Mountain to Lake Perris

Design. This reach completed the Santa Ana Valley Pipeline. It was 12 miles long and constructed entirely of PCCP (Figure 297). The pipe was designed to be 9-foot diameter upstream of the previously installed Riverside Raceway reach and 10-foot diameter downstream from the Raceway. This design used larger pipe in the lower pressured reaches of the Aqueduct for economy (Figure 273). Heads on the 9-foot pipe vary from 200 to 750 feet and from 100 to 300 feet on the 10-foot size.

Bids were invited on alternative bidding schedules as before and, again, bids were received only on the PCCP alternative. Required pipeline appurtenances included eight nozzles with combination air release-vacuum valves, one turnout, and three blowoffs.

The single turnout in this reach was a 108-inch connection located near Box Springs Road. This turnout required selection of the high elevation in the Pipeline just upstream of the Lake Perris intake to provide sufficient head at Box Springs for delivery of 500 cfs.

Three major blowoffs were designed. A 12-inch-diameter blowoff at Station 550+84 (Figure 262) was located at the intersection of Watkins Drive and Big Springs Road. It used a special butterfly valve discharging into a 72-inch-diameter, baffled, concrete-encased, steel pipe designed to function as an energy dissipator before releasing flow into a municipal storm drain. A 12-inch-diameter blowoff at Station 652+60 was designed to discharge into a natural drainage channel which was protected with grouted riprap. The largest blowoff from the 10-foot-diameter pipe was designed as a 16-inch-diameter blowoff to discharge directly into an existing concrete-lined flood channel. All blowoffs from this reach of Santa Ana Valley Pipeline were connected at the top of the pipe and, since these would not fully dewater the pipe, blind, flanged connections were provided at special locations for installation of portable submersible pumps to complete dewatering.

Butterfly valves were specified for both guard valves and flow-regulating valves. The regulating valves were provided with hoods at the ends of the blowoff pipes to improve the flow characteristics. Two-inch bypass lines across the guard valves were designed to allow blowoff piping to be filled before opening a guard valve.

Construction. Open-country pipeline excavation typically was begun with shallow cuts made by bulldozers pushing earth material to the side. Scrapers completed the excavation, where possible, but backhoes were used in harder materials. Drilling and blasting were required at three locations: near Sugarloaf Mountain, near Box Springs Mountain, and at the end of the contract near Lake Perris (Figure 298). Fine trench grading was done by small tractor-dozers and backfilling work generally was mechanized (Figure 299).

About 6,000 feet of this reach of pipeline, which lays within Riverside city streets, was specified to require shored pipe trenches. Excavation in this section was by a backhoe and pipelaying operations followed closely. Fine grading was done by hand, inside a steel trench shield (Figure 300).

As work in the shored trench location proceeded south along Valencia Hill Drive, timber shoring with hydraulically controlled shore bracing was used. This method was successful in reasonably firm ground but was unsatisfactory in sandy material. Sandy soil originally was supported by use of H-piling set in drilled holes with timber sheeting set between the flanges. This procedure proved too slow and expensive, so it was replaced by a movable steel shield pulled along by the backhoe. The shield was satisfactory for protection of personnel, but adjacent property was damaged due to caving trench walls after the shield was moved. However, since the shield doubled the pipelaying progress, it was used for the remainder of the shored trench work.

In the open-cut trench area, the pipe was stored along the trench and placed without difficulty. The maximum placement rate was installation of 54 pipe sections in an 8-hour day. In the shored trench area, pipe placement averaged about three sections of pipe per day with timber shoring and about six sections per day using the shield (Figures 301, 302, and 303).

Native material excavated from the pipe trenches was tried unsuccessfully for use as consolidated backfill. This material was a decomposed granite in which particle cementation and bonds were broken by the vibration effort, resulting in material below the specified grading limitations. Also, since the granite particles were angular, this material did not move well under normal vibration effort, and an excessive amount of jetting water was required to achieve specified consolidation. This method soon was abandoned and imported Santa Ana River sand used, which proved acceptable.

The casing required to pass under the U.S. Highway 60 freeway was installed with tunneling and liner plate. Mining excavation used both tractor-mounted and hand-held air tools. Ground water was encountered in this tunnel, but it was drained easily to a pump sump at the low end (Figure 304).

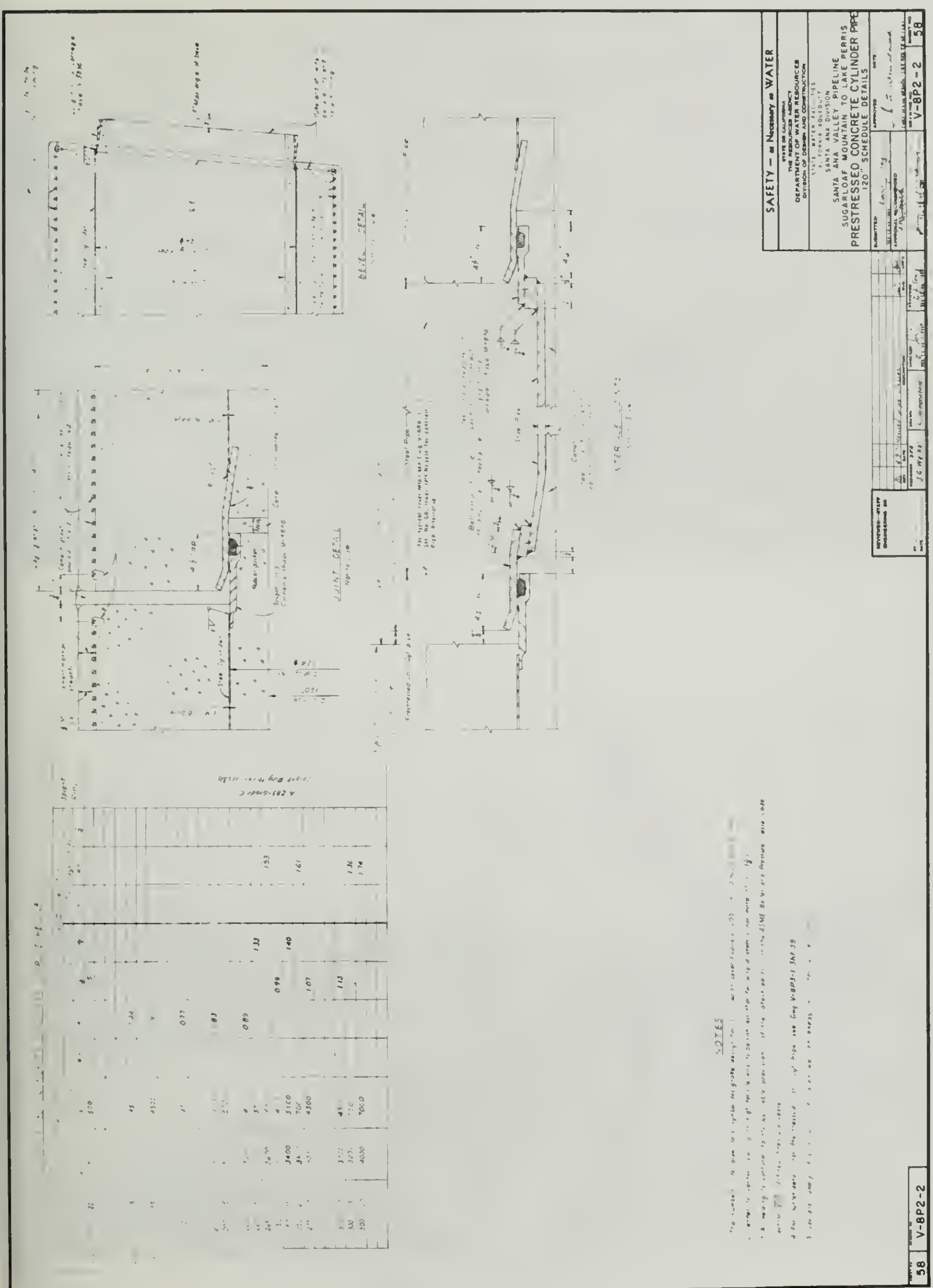


Figure 297. Prestressed-Concrete Cylinder Pipe Details—Sugarloaf Mountain to Lake Perris

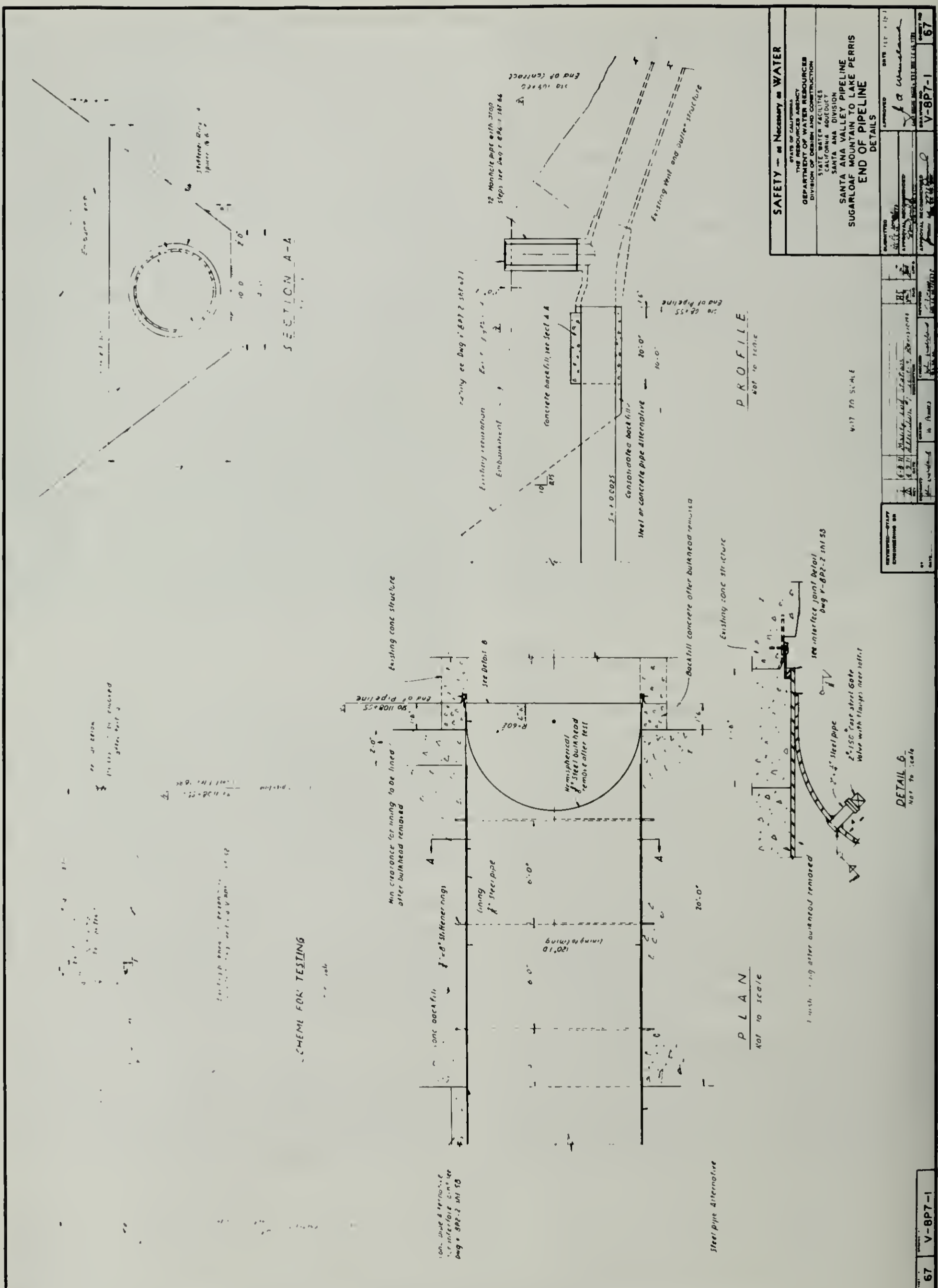


Figure 298. End of Pipeline Details

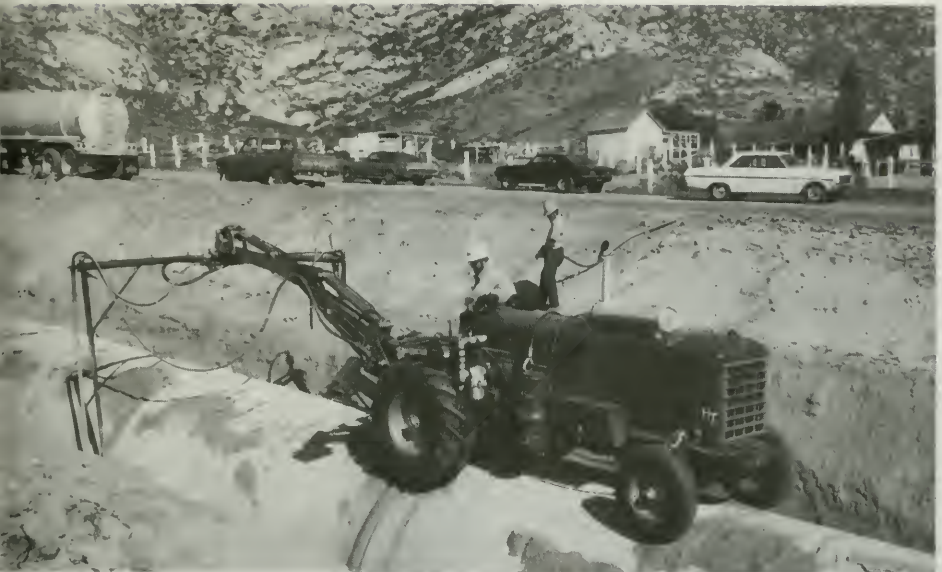


Figure 299. Vibrating Backfill far Consalidatian

Figure 300. Pipeline Construction in Urban Area

Figure 301. Pipe in Trench Along Watkins Drive in Riverside

Figure 302. Caved Trench Wall

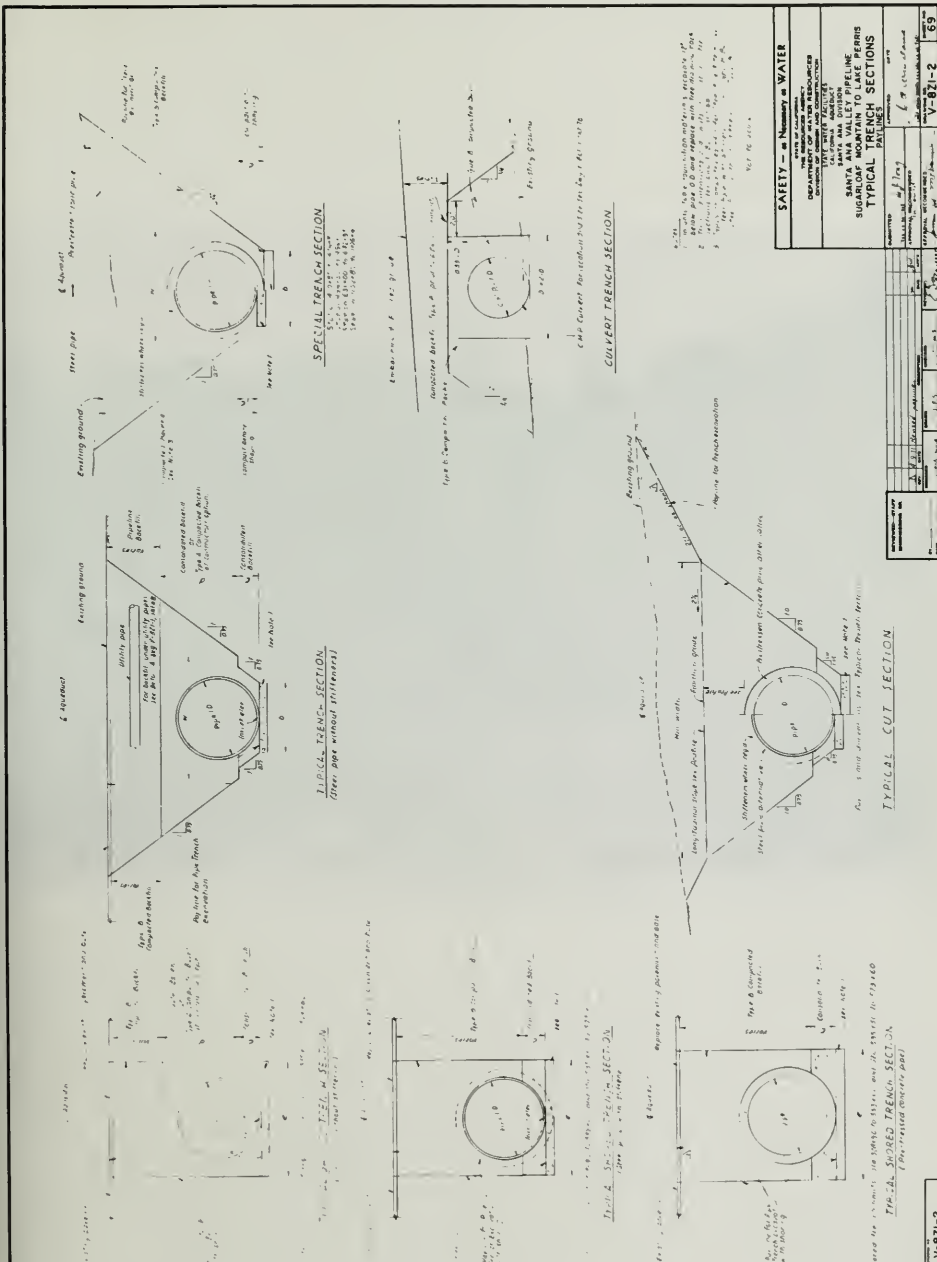


Figure 303. Typical Trench Sections—Sugarloaf Mountain to Lake Perris

Figure 304. Tunnel Under U.S. Highway 60 Freeway

APPENDIX A

ENGLISH TO METRIC CONVERSIONS

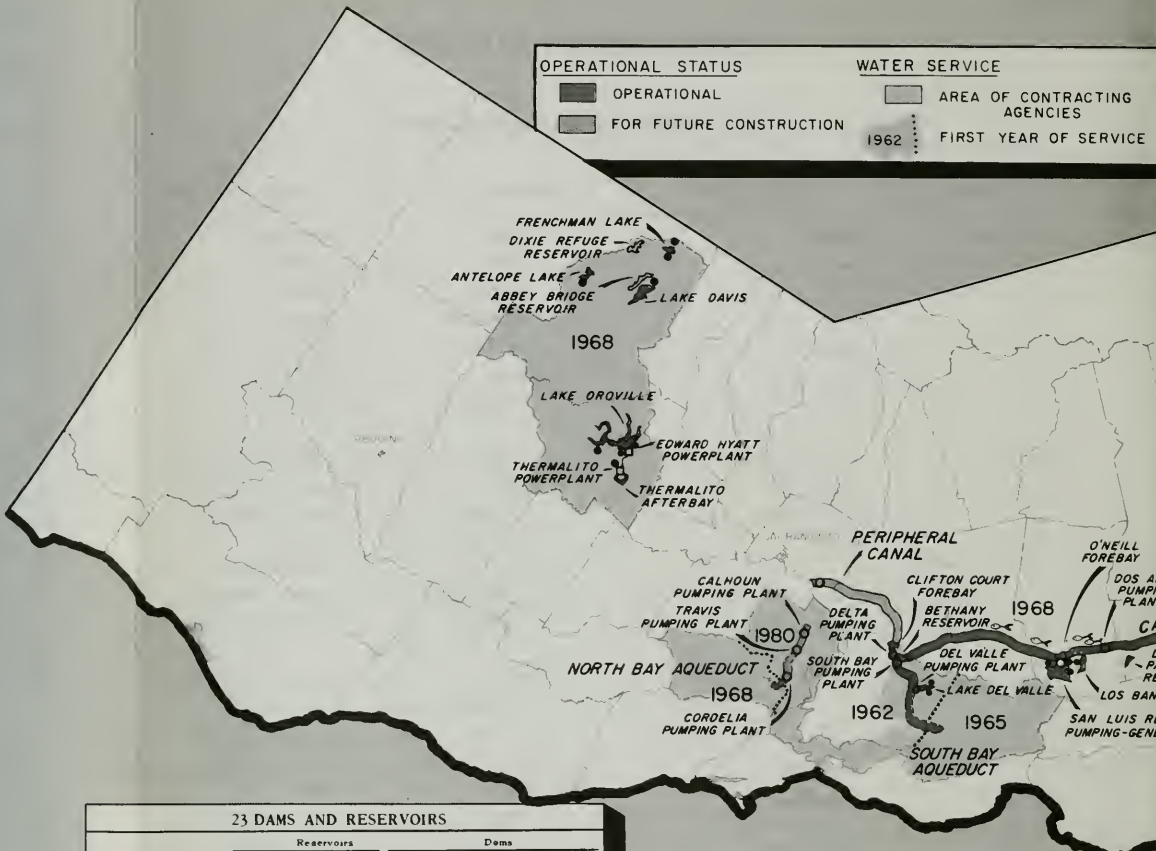
AND PROJECT STATISTICS

CONVERSION FACTORS

English to Metric System of Measurement

Quantity	English unit	Multiply by	To get metric equivalent
Length	inches	2.54	centimeters
	feet	30.48	centimeters
		0.3048	meters
		0.0003048	kilometers
	yards	0.9144	meters
	miles	1,609.3	meters
		1.6093	kilometers
Area	square inches	6.4516	square centimeters
	square feet	929.03	square centimeters
	square yards	0.83613	square meters
	acres	0.40469	hectares
		4,046.9	square meters
		0.0040469	square kilometers
	square miles	2.5898	square kilometers
Volume	gallons	3,785.4	cubic centimeters
		0.0037854	cubic meters
		3.7854	liters
	ocre-feet	1,233.5	cubic meters
		1,233,500.0	liters
	cubic inches	16.387	cubic centimeters
	cubic feet	0.028317	cubic meters
	cubic yards	0.76455	cubic meters
		764.55	liters
Velocity	feet per second	0.3048	meters per second
	miles per hour	1.6093	kilometers per hour
Discharge	cubic feet per second or second-feet	0.028317	cubic meters per second
Weight	pounds	0.45359	kilograms
	tons (2,000 pounds)	0.90718	tons (metric)
Power	horsepower	0.7460	kilowatts

PROJECT



23 DAMS AND RESERVOIRS

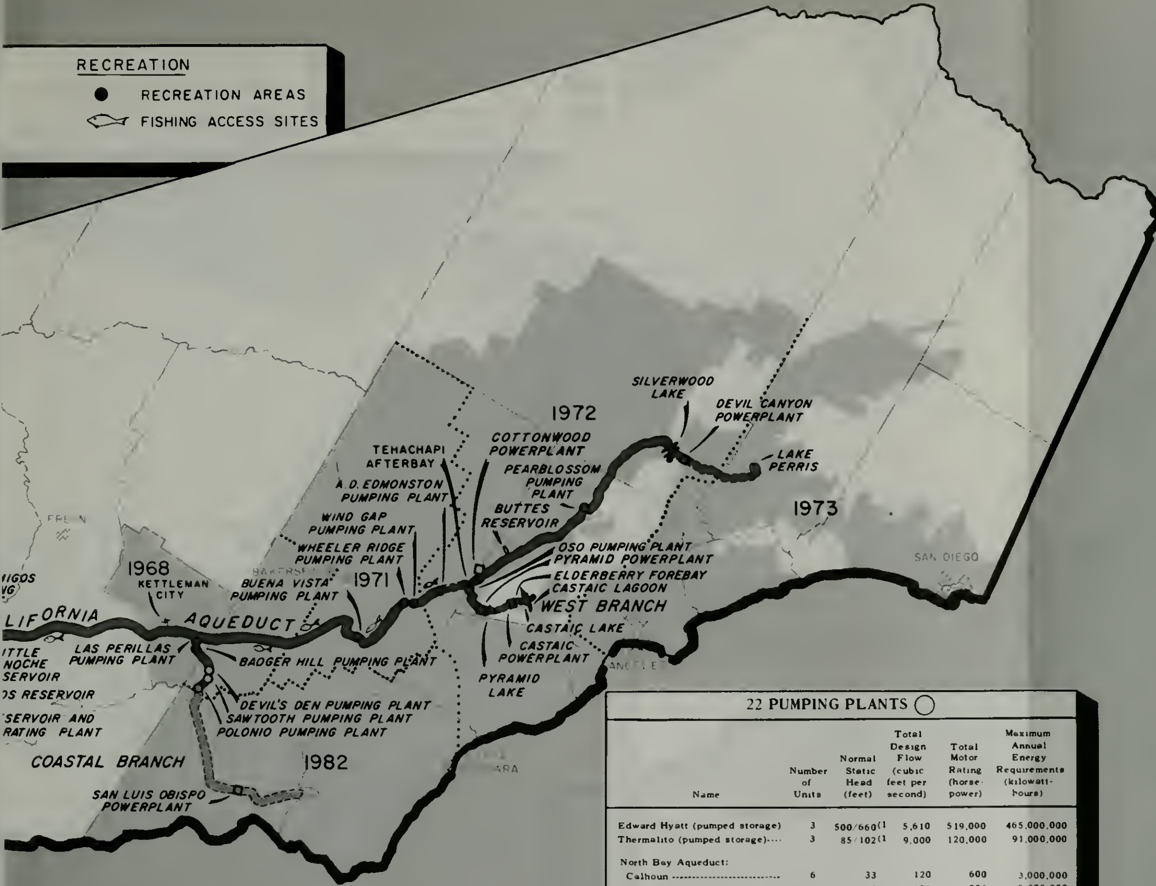
Name of Reservoir	Reservoirs: Gross Capacity(1 (Acre-feet)	Reservoirs: Surface Area (acres)	Reservoirs: Shore-line (miles)	Dams: Structural Height (feet)	Dams: Crest Elevation(2 (feet)	Dams: Crest Length (feet)	Dams: Volume (cubic yards)
Frenchman Lake	55,477	1,580	21	139	5,607	720	537,000
Antelope Lake	22,566	931	15	120	5,025	1,320	380,000
Lake Davis	84,371	4,026	32	132	5,785	800	253,000
Abbey Bridge	45,000	1,950	21	117	5,4[illegible]	1,150	500,000
Dixie Refuge	16,000	900	15	100	5,754	1,050	400,000
Lake Oroville	3,537,577	15,805	167	770	922	6,920	80,000,000
Thermalito Diversion Pool	13,328	323	10	143	233	1,300	154,000
Fish Barrier Pool	580	52	1	91	181	600	10,500
Thermalito Forebay	11,768	630	10	91	231	15,900	1,840,000
Thermalito Afterbay	57,041	4,302	26	39	142	42,000	5,020,000
Clifton Court Forebay	28,653	2,109	8	30	14	36,500	2,440,000
Bethany	4,804	161	6	121	250	3,940	1,400,000
Lake Del Valle	77,106	1,060	16	235	773	880	4 150,000
San Luis	2,038,771	12,700	65	385	554	18,600	77,645,000
O'Neill Forebay	56,426	2,700	12	88	233	14,350	3,000,000
Los Banos	34,562	623	12	167	384	1,370	2,100,000
Little Panoche	13,236	354	10	152	676	1,440	1,210,000
Buttes	21,800	580	6	190	2,790	2,230	3,130,000
Silverwood Lake	74,970	976	13	249	3,378	2,230	7,600,000
Lake Perris	131 452	2,318	10	128	1,600	11,600	20,000,000
Pyramid Lake	171,196	1,297	21	400	2,606	1,090	6,860,000
Elderberry Forebay	28,231	460	7	200	1,550	1,990	6,000,000
Castaic Lake	323,702	2,235	29	425	1,535	4,900	46,000,000
Totals	6,848,617	58,072	533			172,880	270,629,500

1) At maximum normal operating level.

2) Above sea level.

AQUEDUCTS

Name	Length (miles): Total	Canal	Pipeline	Tunnel	Channel and Reservoir
North Bay Aqueduct	26.5	14.3	12.2	0	0
South Bay Aqueduct	42 9	6.4	32 9	1.6	0
Peripheral Canal	43.0	42.0	1.0	0	0
Subtotal	112.4	64.7	46.1	1.6	0
California Aqueduct (main line):					
Delta to O'Neill Forebay	68.4	67.0	0	0	1.4
O'Neill Forebay to Kettleman City	105.7	103.5	0	0	2.2
Kettleman City to A. D. Edmonston Pumping Plant	120 9	120.9	0	0	0
A. D. Edmonston Pumping Plant thru Tehachapi Afterbay	10 6	0.2	2.5	7.9	0
Tehachapi Afterbay thru Lake Perris	136 4	93.4	36.3	3.8	2.9
Subtotal, main line	444.0	365 0	40.6	11.7	6 5
California Aqueduct (branches):					
West Branch	31.9	9.1	6 4	7.2	9 2
Coastal Branch	96 2	14 6	81 4	0	0
Subtotal, branches	128.1	23 9	67 8	7.2	9.2
Totals	664.5	473.6	174 7	20 5	15 7



8 POWERPLANTS

Name	Number of Units	Normal Static Head (feet)	Total Design Flow (cubic feet per second)	Power Generator Output (kilowatts)	Maximum Annual Energy Output (kilowatt-hours)
Edward Hyatt	6	410/676[1]	14,550	678,750	2,475,000,000
Thermalito	4	85/100[1]	16,900	119,600	383,000,000
San Luis					
Total	8	99/327[1]	13,120	424,000	
State Share			6,872	222,100	170,000,000
Cottonwood	1	140	1 637	15,000	115,000 000
Devil Canyon	2	1,418	1 200	119,700	1,003,000,000
Pyramid	2	740	3,100	157,000	1,001,000,000
Castaic					
Total	7	1,063	18 400	1,250,000	
State Share[2]			3,092	214,000	1,457,000,000
San Luis Obispo	1	730	111	5,900	41,000,000
Total, State Share					6,645,000,000

1) Minimum and maximum static heads

2) The City of Los Angeles Department of Water and Power will construct and operate a 1,250,000-kilowatt Castaic Powerplant and will supply the Project with electrical power and energy equivalent to the generation from a 213,984-kilowatt powerplant the State originally planned to construct

22 PUMPING PLANTS

Name	Number of Units	Normal Static Head (feet)	Total Design Flow (cubic feet per second)	Total Motor Rating (horsepower)	Maximum Annual Energy Requirements (kilowatt-hours)
Edward Hyatt (pumped storage)	3	500/660[1]	5,610	519,000	465,000,000
Thermalito (pumped storage)	3	85/102[1]	9,000	120,000	91,000,000
North Bay Aqueduct:					
Calhoun	6	33	120	600	3,000,000
Travis	6	0	120	900	5,000,000
Cordelia	3	448	48	3,100	14,000,000
South Bay Aqueduct:					
South Bay	9	545	330	27,750	166,000,000
Del Valle	4	0/38[2]	120	1,000	2,000,000
California Aqueduct (main line):					
Delta	11	244	10,303	333,000	1,355,000,000
San Luis					
Total	8	99/327[2]	11,000	504,000	
State Share			5,762	264,000	313,000,000
Dos Amigos					
Total	6	113	13,200	240,000	
State Share			7,100	130,000	607,000,000
Buena Vista	10[3]	205	5,049	136,000	746,000,000
Wheeler Ridge	9[3]	233	4,598	140,000	797,000,000
Wind Gap	9[3]	518	4,410	308,000	1,761,000,000
A. D. Edmonston	14[3]	1,926	4,095	1,040,000	5,916,000,000
Pearblossom	6	540	1,380	113,200	647,000,000
California Aqueduct (branches):					
Oso	8	231	3,128	93,800	446,000,000
Las Perillas	6	55	450	4,050	20,000,000
Badger Hill	6	151	450	10,500	56,000,000
Devil's Den	4	409	126	8,000	51,000,000
Sawtooth	4	331	126	6,500	41,000,000
Polonio	4	810	126	16,000	101,000,000
Peripheral Canal					
Total	9[3]	10	21,800	35,200	
State Share			10,900	17,440	88,000,000
Total, State Share					13,691,000,000

1) Minimum and maximum total pumping heads.

2) Minimum and maximum static heads.

3) Includes one spare unit.

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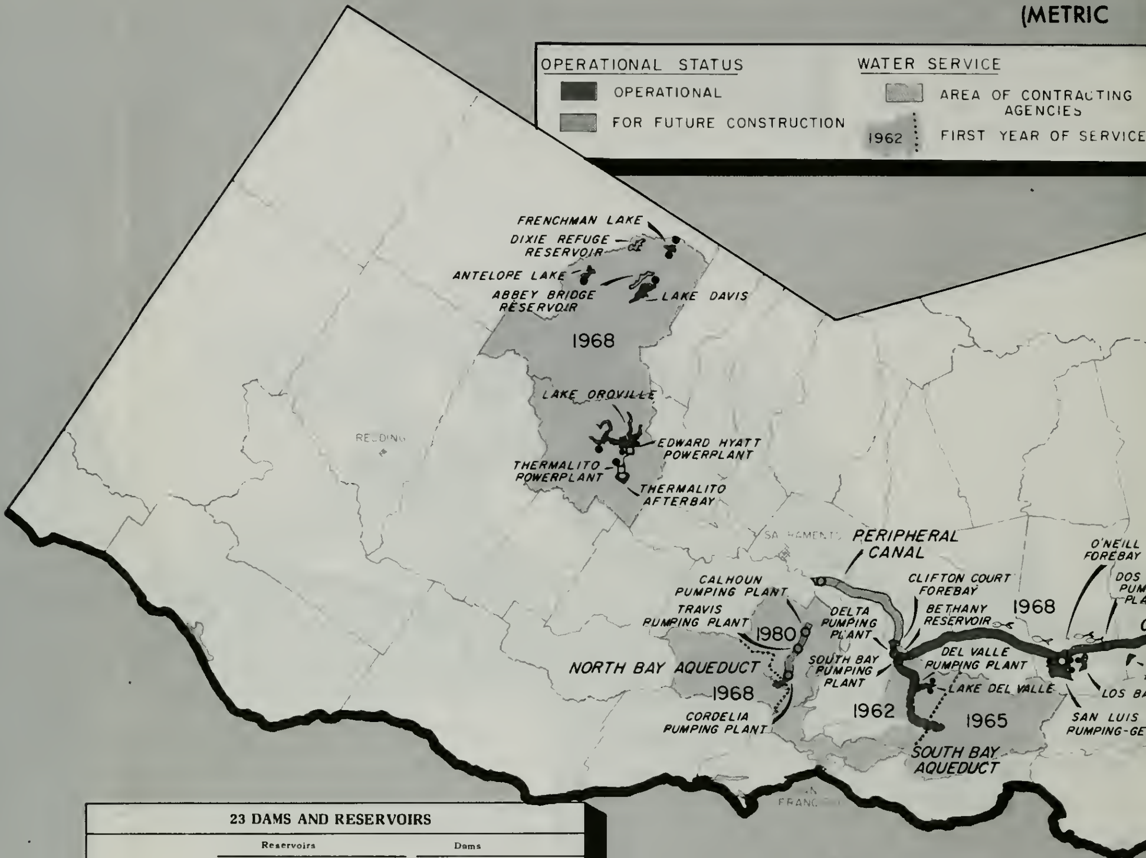


23 DAMS AND RESERVOIRS

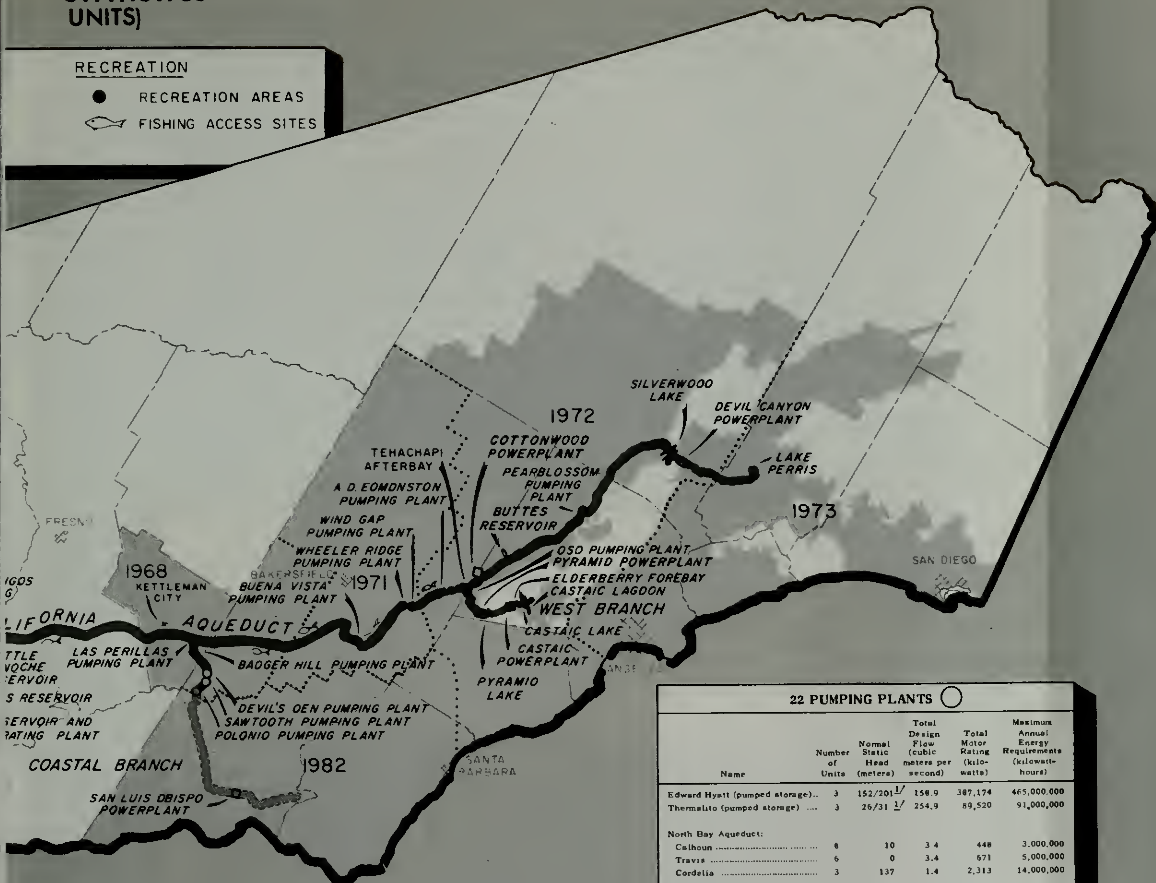
Name of Reservoir	Reservoirs: Gross Capacity[1] (millions of cubic meters)	Reservoirs: Surface Area (hectares)	Reservoirs: Shoreline (kilometers)	Dams: Structural Height (meters)	Dams: Crest Elevation[2] (meters)	Dams: Crest Length (meters)	Dams: Volume (cubic meters)
Frenchman Lake	68.43	639	33.8	42	1709	219	410,600
Antelope Lake	27.84	377	24.1	37	1532	402	290,500
Lake Davis	104.07	1,629	51.5	40	1763	244	193,400
Abbey Bridge	55.51	789	33.8	36	1669	351	382,300
Dixie Refuge	19.74	364	24.1	30	1754	320	305,800
Lake Droville	4,363.60	6,396	268.8	235	281	2,109	61,164,000
Thermalito Diversion Pool	16 44	131	16 1	44	71	396	117,700
Fish Barrier Pool	0.72	21	1.6	28	55	183	8,000
Thermalito Forebay	14.52	255	16.1	28	70	4,846	1,406,800
Thermalito Afterbay	70.36	1,741	41.8	12	43	12,802	3,838,000
Clifton Court Forebay	35.34	853	12.9	9	4	11,125	1,865,500
Bethany	5.93	65	9.7	37	76	1,201	1,070,300
Lake Del Valle	95.11	429	25.8	72	236	268	3,172,900
San Luis	2,514.82	5,140	104.6	117	169	5,669	59,363,500
O'Neill Forebay	69.60	1,093	19.3	27	71	4,374	2,293,700
Los Banos	42 63	252	19.3	51	117	418	1,605,600
Little Panoche	16 33	143	16 1	46	206	439	925,100
Buttes	26.89	235	9.7	58	850	680	2,393,000
Silverwood Lake	92.48	395	20.9	76	1,030	680	5,810,600
Lake Perris	162.15	938	16 1	39	488	3,536	15,291,000
Pyramid Lake	211.17	525	33.8	122	794	332	5,244,800
Elderberry Forebay	34 82	186	11 3	61	472	607	4,587,300
Castaic Lake	399 29	904	46.7	130	468	1,494	35,169,300
Totals	8,447.79	23,500	857.9			52,695	206,909,700

1/ At maximum normal operating level.
2/ Above sea level.

AQUEDUCTS

Name	Length (kilometers): Total	Canal	Pipeline	Tunnel	Channel and Reservoir
North Bay Aqueduct	42.6	23 0	19 6	0	0
South Bay Aqueduct	69 1	13.5	53.0	2.6	0
Peripheral Canal	69.2	67 6	1 6	0	0
Subtotal	180.9	104.1	74.2	2.6	0
California Aqueduct (main line):					
Delta to O'Neill Forebay	110.1	107.8	0	0	2.3
O'Neill Forebay to Kettleman City	170 1	166 6	0	0	3.5
Kettleman City to A.D. Edmonston Pumping Plant	194.6	194 6	0	0	0
A.D. Edmonston Pumping Plant thru Tehachapi Afterbay	17 0	0 3	4 0	12 7	0
Tehachapi Afterbay thru Lake Perris	222 7	150.3	61 6	6 1	4 7
Subtotal, main line	714.5	619.6	65.6	18.8	10.5
California Aqueduct (branches):					
West Branch	51.3	14.6	10.3	11.6	14.8
Coastal Branch	154.8	23.8	131.0	0	0
Subtotal, branches	206 1	38.4	141.3	11.6	14 8
TOTALS	1,101.5	762.1	281 1	33 0	25.3

STATISTICS
UNITS)



8 POWERPLANTS

Name	Number of Units	Normal Static Head (meters)	Total Design Flow (cubic meters per second)	Power Generator Output (kilowatts)	Maximum Annual Energy Requirements (kilowatt-hours)
Edward Hyatt	6	125/206 1/	412.0	678,750	2,475,000,000
Thermalito	4	26/30 1/	478.6	119,600	383,000,000
San Luis					
Total	8	30/100 1/	371.6	424,000	
State Share			194.6	222,100	170,000,000
Cottonwood	1	43	46.4	15,000	115,000,000
Devil Canyon	2	432	34.0	119,700	1,003,000,000
Pyramid	2	226	87.8	157,000	1,001,000,000
Castaic					
Total	7	324	521.0	1,250,000	
State Share 2/			87.6	214,000	1,457,000,000
San Luis Obispo	1	223	3.1	5,900	41,000,000
Total, State Share					6,645,000,000

1/ Minimum and maximum static heads.

2/ The City of Los Angeles Department of Water and Power will construct and operate a 1,250,000-kilowatt Castaic Powerplant and will supply the Project with electrical power and energy equivalent to the generation from a 213,984-kilowatt powerplant the State originally planned to construct.

22 PUMPING PLANTS

Name	Number of Units	Normal Static Head (meters)	Total Design Flow (cubic meters per second)	Total Motor Rating (kilowatts)	Maximum Annual Energy Requirements (kilowatt-hours)
Edward Hyatt (pumped storage)	3	152/201 1/	158.9	387,174	465,000,000
Thermalito (pumped storage)	3	26/31 1/	254.9	89,520	91,000,000
North Bay Aqueduct:					
Calhoun	8	10	3 4	448	3,000,000
Travis	6	0	3.4	671	5,000,000
Cordelia	3	137	1.4	2,313	14,000,000
South Bay Aqueduct:					
South Bay	9	166	9.3	20,702	166,000,000
Del Valle	4	0/12 2/	3.4	746	2,000,000
California Aqueduct (main line):					
Delta	11	74	291.8	248,418	1,355,000,000
San Luis					
Total	8	30/100 2/	311.5	375,984	
State Share			163.2	196,944	313,000,000
Dos Amigos					
Total	6	34	373.8	179,040	
State Share			201 1	96,980	607,000,000
Buena Vista	10 3/	62	143.0	101,456	746,000,000
Wheeler Ridge	9 3/	71	130.2	104,440	797,000,000
Wind Gap	9 3/	158	124.9	229,768	1,761,000,000
A.D. Edmonston	14 3/	587	116.0	775,840	5,916,000,000
Pearblossom	6	165	39 1	84,447	647,000,000
California Aqueduct (branches):					
Oso	8	70	88 6	69,975	446,000,000
Las Perillas	6	17	12.7	3,021	20,000,000
Badger Hill	6	46	12.7	7,835	56,000,000
Devil's Den	4	125	3.6	5,968	51,000,000
Sawtooth	4	101	3.6	4,849	41,000,000
Polonio	4	247	3 6	11,936	101,000,000
Peripheral Canal					
Total	9	3	617.3	26,259	
State Share			308 7	13,010	88,000,000
Total, State Share					13,691,000,000

1/ Minimum and maximum total pumping heads.

2/ Minimum and maximum static heads.

3/ Includes one spare unit.

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